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PRESSURES ASSOCIATED WITH SWELLING IN SOILS

by

JAMES GORIS

A THESIS

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MAY, 1965

UNIVERSITY OF ALBERTA
FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read,
and recommend to the Faculty of Graduate Studies for
acceptance a thesis entitled "PRESSURES ASSOCIATED WITH
SWELLING IN SOILS", submitted by JAMES GORIS in partial
fulfilment of the requirements for the degree of Master
of Science.

Date May 18, 1965

ABSTRACT

This thesis concerns a laboratory study of pressures associated with swelling in soils. The nature of swelling pressures in undisturbed samples of a highly overconsolidated clay shale was studied under controlled laboratory conditions.

A specially designed consolidation cell was constructed. This cell employed a pressure transducer to measure pore-water pressures during critical periods of the swelling tests. The cell also allowed control of the chemical nature of the solution allowed to the soil during the tests.

A total of 10 tests was performed. Four standard free-swell tests in which solutions of distilled water, 0.2, 0.4, and 0.6 normal sodium chloride were used as the immersing fluid determined the effect of various salt concentrations on the free-swell swelling pressure. The effect of these salt concentrations on the amount of free swell was also studied. Four standard constant-volume swell tests using identical solutions as in the free-swell tests determined the effect of different salt concentrations on constant-volume swelling pressures. Two tests performed in the special consolidation cell determined the effect on internal pressures of changing the salt concentration of the immersing fluid during a constant-volume swell test.

The results of the free-swell tests indicated that the salt concentration of the immersing fluid had a considerable effect on the free-swell swelling pressure and on the amount of free swell that occurred.

The results of the constant-volume swell tests indicated that various salt concentrations also affect constant-volume swelling pressures, but that these swelling pressures were highly dependent on small amounts of volume changes that occurred.

The special consolidation tests indicated that changes in the salt concentration of the solution allowed to a soil during constant volume swelling results in the development of an osmotic pressure between the pore water in the soil and the immersing fluid. These tests also indicated that an increase in total stress on the soil after changes in the salt concentration of the immersing fluid resulted in no measurable pore-water pressures.

From the results of this investigation, it is recommended that research on the same basic problem be continued. It is also recommended that a study be undertaken to determine the exact differences between free-swell swelling pressures and constant-volume swelling pressures.

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to Dr. E.W. Brooker for supervising the experimental research presented in this manuscript. He gratefully acknowledges his especial debt to Dr. R.M. Hardy for initially suggesting the project and offering greatly appreciated advice. Thanks are given to Professor P.H. Bouthillier for conducting the chemical analyses of the pore water in the soil.

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GLOSSARY OF TERMS AND SYMBOLS

TERMS

ADSORBED WATER..... The Water in a soil mass that is held by physico-chemical forces to the surface of a soil particle. It is considered to have physical properties different from "ordinary" water when both are at the same temperature and pressure.

CATION EXCHANGE CAPACITY..... The quantity of cations that can be adsorbed by a clay soil. It is usually measured in milliequivalents per 100 grams of air dry soil.

COMPRESSIBILITY OF APPARATUS..... The amount of compression, measured in inches, that occurs when the consolidation apparatus is loaded without soil present.

CONSTANT-VOLUME SWELL TEST..... A one-dimensional consolidation test in which a soil sample is maintained at constant volume by the addition of small load increments.

ELECTROLYTE CONCENTRATION..... The concentration of soluble salts in a water solution expressed as normality.

FREE-SWELL TEST..... A one-dimensional consolidation test in which the soil sample is allowed free access to water while under a small surcharge.

HOMIONIC SOIL..... A soil in which all the exchange positions of the clay mineral are occupied by a single cation species.

IMMERSING FLUID..... The water solution allowed to a soil sample in a free-swell test or constant-volume swell test.

LEACHING PROCESS..... The process by which the water solution present in the porous center plate is replaced by a different solution.

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LEACHING PROCESS..... The process by which the water solution present in the porous center plate is replaced by a different solution.

NORMAL SOLUTION..... A water solution containing one gram-equivalent weight per 1000 milliliters of water.

OSMOTIC PRESSURE..... The hydrostatic pressure created by two solutions of different concentrations separated by a semi-permeable membrane. It is postulated to act between the double layer water and the free pore water in the soil voids.

OVERCONSOLIDATED SOIL..... A soil deposit that has been subjected to a past overburden pressure greater than the present overburden pressure.

PHYSICO-CHEMICAL..... The influence of physical and chemical forces on soil behavior.

PORE WATER..... All free water in a clay water system excluding adsorbed water.

PRINCIPLE OF EFFECTIVE STRESS..... A fundamental concept of soil mechanics which states that all volume changes such as compression, distortion, or shear are caused by changes in the effective stress. The effective stress is usually the numerical difference between total stress and pore-water pressure.

SHEARING RESISTANCE

SHEAR STRENGTH..... The maximum resistance a soil can offer to shearing stresses.

SALT CONCENTRATION..... The concentration of soluble salts in a solution expressed as normality.

VOLUME CHANGE..... A change in the total volume of a soil. For the test results in this thesis, a change in the height of a soil sample as measured by a vertical dial indicator.

SYMBOLS

C'	...	Effective Cohesion Intercept.
P_s	...	Swelling Pressure as measured from a conventional laboratory swell test.
ϕ'	...	Effective angle of shearing resistance.
U	...	Pore Water Pressure.
Π	...	Osmotic Pressure.
σ'	...	Effective normal stress.
σ	...	Total normal stress.
T_f	...	Shearing resistance

CHAPTER I

INTRODUCTION

1.1 General

Many natural deposits of swelling type clay soils have been encountered throughout Western Canada and in some parts of the United States. A large portion of these soils have been overconsolidated in their geologic history either by glaciation or by desiccation, but are presently under substantially reduced overburden pressures. These soils, having undergone a stress reduction often display highly expansive properties and are capable of exerting free-swell swelling pressures in excess of the existing overburden pressure (Hardy & Dyregrov, 1962). A variety of earthwork and foundation problems exist in swelling soil types. Examples of such problems are the heave of spillway structures (Peterson & Peters, 1963) and slope stability (Hardy & Dyregrov, 1962). Experience has shown that these problems are presently beyond quantitative analysis.

The problems associated with expansive soils may be classified into two principal groups. One group includes those associated with large volume changes caused by changes in moisture content.

Damage done to light buildings placed on shallow foundations in swelling clays, damage to highway pavements in the form of heave, uplifting of sidewalks, and cracking of exterior walls, are examples of this group of problems (Means, 1959; Dawson, 1959). In those cases where swelling soils are involved, the problems were initiated because of volume changes taking place during wetting and drying cycles in the history of the soil.

A second group of problems associated with expansive soils is concerned with changes that may take place in shearing resistance when the physico-chemical condition of the soil is altered. This group of problems has been investigated in terms of physico-chemical phenomena and is approached from a microscopic direction. Researchers such as Rosenqvist & Bjerrum (1957), Lambe (1959), Thomson (1963), and Hardy (1964) have shown that shear strength can be plausibly explained for certain types of swelling soil when approached from a physico-chemical point of view. Both field and laboratory research concerned with the mechanisms of these problems have been underway for about 25 years, with the most significant advances being made during the past decade.

1.2 Statement of Problem

It is universally accepted that the principle of effective stress governs the shearing resistance of soils. The concept of effective stress states that "all measurable

effects of a change of stress, such as compression, distortion, and a change of shearing resistance are exclusively due to changes in the effective stresses (Terzaghi, 1925)." The principle of effective stress has been applied to the shearing resistance of a soil by means of a modified form of Coulomb's law:

$$T_f = C' + (\sigma - U) \tan \phi' \quad 1.1$$

where T_f is the shearing resistance, C' is the effective cohesion intercept, σ is the normal stress, U is the pore-water pressure, and ϕ' is the effective angle of shearing resistance. This concept has proven applicable to a wide variety of soils (Skempton, 1961). In swelling soils, the application of this principle is less obvious. Lambe (1959) concludes that the effective stress principle does not satisfactorily correlate with the observed behavior of swelling clays. His "mechanistic approach" to the principle leads to the conclusion that it is very difficult to define effective stress whenever a large portion of the water contained within a soil is strongly attracted to the surfaces of the clay particles. Soils with the greatest swelling tendencies fall into this category. Based upon experience with stability problems in the Alberta area, Hardy et al, (1962) have proposed a modification of Coulomb's law in terms of effective stress to account for reductions in shearing resistance

associated with failure conditions in swelling soil types.

Thus:

$$T_f = C' + (\nabla - U - P_s) \tan \phi' \quad 1.2$$

where C' , ∇ , U , ϕ' , and T_f are as before, and P_s is the swelling pressure.

This modification suggests that the effective stress as usually determined by equation 1.1 might be reduced by an additional amount equal to a swelling pressure as determined from a conventional consolidation test. The modification is primarily based on investigations of two landslides which occurred in clay-shales of Upper Cretaceous age. These shales are highly pre-consolidated (Peterson & Peters, 1963) and are known to be of an expansive nature (Hardy et al, 1962). When the conventional form of Coulomb's law, using maximum effective strengths as determined by consolidated-undrained triaxial tests with pore pressure measurements, was applied to these soils, the stability analyses yielded factors of safety consistently greater than unity. Reducing the effective stress by an amount equal to the swelling pressure yielded factors of safety which were more consistent with observed behavior. Whether or not this modification is theoretically or experimentally justifiable has not yet been determined. Therefore, an experimental study of the causes of swelling is believed to be of significance.

A knowledge of changes in volume or of shearing resistance that may take place with changes in the chemical or physical environment of a highly swelling soil has an application in a wide variety of engineering problems, particularly those problems encountered in highly over-consolidated soils. The present study is concerned with one aspect of this problem.

1.3 Purpose of Investigation

The purpose of this study is to examine the influence of variations in chemical and physical environment on effective stresses developed during a one-dimensional compression test on a swelling soil. It has three objectives:

1. To investigate the nature of swelling pressures and more clearly define their application to swelling soils.
2. To obtain data which would indicate the validity of the hypothesis proposed by equation 1.2.
3. To indicate the direction that future research might follow.

The tests were conducted in a specially designed one-dimensional consolidation cell. This cell allowed measurements of total stress and of pore water pressure during controlled alteration of both chemical and physical environment.

The first part of the paper discusses the importance of the study of the history of the English language. It is argued that the study of the history of the English language is not only a matter of historical interest, but also a matter of practical importance. The study of the history of the English language is essential for the understanding of the English language in its present state. It is also essential for the understanding of the English language in its future state. The study of the history of the English language is also essential for the understanding of the English language in its present and future state.

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1.4 Scope of Study

This study includes the results of two special consolidation tests during which pore pressures were measured as the salt concentration of the leaching fluid was varied. The soil was obtained from the South Saskatchewan River Dam in the summer of 1964. Previous investigations of this soil were conducted by Peterson & Peters (1963). Stratigraphically it is termed Bearpaw Shale.

In addition to the special consolidation tests, constant-volume and free-swell tests were performed to determine the swelling properties of the material. Soil classification tests in the form of specific gravity, Atterberg limits, and grain size classification tests were performed to determine the soil properties. Composition of the pore water, clay mineralogy, and cation exchange capacities were also determined to evaluate the physico-chemical properties. The major part of the research was conducted in the soil mechanics laboratory, Civil Engineering Department, University of Alberta.

The following chapter deals with the attack on the problem, the methods of measuring the variables involved, the laboratory equipment employed, and the general test procedures.

Chapter III presents the results of the study.

A discussion and interpretation of the test results in relation to present day knowledge is presented in Chapter IV.

The investigation is summarized and conclusions from it are given in the final chapter. Recommendations for future research are made.

CHAPTER II

EXPERIMENTAL PROCEDURE

2.1 Introduction

The purpose of this chapter is to describe the attack on the problem, the method of measuring the variables involved, the laboratory equipment employed, and the general test procedures. Similar to any research program, it was desirable and necessary to eliminate all variables other than those under investigation.

Considerable time and effort were spent on the design and construction of a special one-dimensional consolidation cell instrumented to measure pertinent stresses developed during the progress of the test. A detailed graphical description of the component parts of this cell is given in Appendix B to this thesis.

2.2 Attack on the Problem

The problem comprising this investigation arose as a result of studies conducted by Hardy, et al, (1962) on highly overconsolidated clay soils of western Canada. From these studies it was concluded that in certain type swelling soils, a variation of chemical environment during a constant-volume swell test results in a decrease in effective stresses.

It was decided to undertake a laboratory research study to examine the validity of this conclusion.

The first step in the attack on the problem was to determine what type of soil would best be suited for this study. It was decided at the outset of the program that the soil to be used should be undisturbed and of a highly swelling nature. Also, it should contain a preponderance of clay-sized minerals with the main clay mineral being montmorillonite, as well as the adsorbed cation being sodium. These soil characteristics were necessary for maximum effects of the physico-chemical action to develop (Thomson, 1963).

The soil chosen was obtained from the south Saskatchewan River Dam in the summer of 1964. The soil had been previously investigated by Peterson & Peters (1963) and was known to meet the necessary experimental requirements. Geologically, the soil is classed as a clay shale of Upper Cretaceous age. It is known to be highly over-consolidated and expands considerably on contact with water (Peterson, 1954).

Soil classification tests were performed on this soil to determine basic soil properties. The tests were performed in accordance with the American Society for Testing and Material and included Specific Gravity (D854-52), Liquid Limit (D423-54T), Plastic Limit (D424-54T), and Shrinkage Limit (D427-39). Triplicate Grain Size Analyses (D422-54T) were performed to determine the grain-size distribution and

and the preponderance of clay-sized particles.

Physico-chemical tests were also performed on this soil to determine its physico-chemical properties. The concentration of soluble salts present in the pore water was determined by Professor P.H. Bouthillier, Civil Engineering Department, University of Alberta. The mineralogical composition of the clay fraction was determined by the Alberta Research Council using X-ray diffraction techniques. Exchangeable cations and total exchange capacity were determined by the Soil Science Department, University of Alberta. From the soil classification the soil is classified as a highly plastic clay. From the physico-chemical tests, it was apparent that the predominant clay mineral was montmorillonite; the exchangeable complex was predominantly absorbed sodium.

The second step in the attack on the problem was to determine the variables involved, the variables which could be controlled, and the variables which might be measured. From the nature of the problem, it was decided that control of the following variables was necessary in order to produce results of a meaningful nature;

1. The total load applied to the sample.
2. The chemical content of water allowed to the soil during the swelling process.
3. The amount of volume change (swell) to take place.

From a study of the physico-chemical action in soils, the following variables were considered to be uncontrollable:

1. The structure of the soil including the orientation of the soil particles.
2. Electrical forces due to particle contacts.

The effect on the test results of these variables could not be evaluated since they could not be controlled. To minimize their effect undisturbed soil samples from the same bore hole and from the same depth (108.6 - 109.1 feet) were used. This point will be discussed fully in a later section of this thesis.

The variables that could be measured were dependent on the type of laboratory test method employed. For this research program both constant-volume swell tests and free-swell tests were conducted. In conjunction with the special consolidation cell the variables considered to be measurable include:

1. The pore-water pressure.
2. The total load applied to the sample.
3. The amount of volume change taking place during the swelling process.

For the constant-volume and free-swell tests performed using standard laboratory equipment, only the total load applied to the soil and the amount of volume change taking place could be measured.

2.3 Method of Measuring Variables

Theoretically, the amount of volume change in a constant-volume swell test is zero. In actual laboratory practice some volume change will occur but the amount is usually considered negligible when compared to that which takes place in a free-swell test. In this research project some volume change took place during the swelling process. The amount taking place was measured by the use of a 0.0001 inch British dial indicator. This measuring device is standard equipment for the laboratory consolidation apparatus.

To control changes in the chemical environment of the soil-water system the special consolidation cell was equipped with a series of zero-volume-change Circle Seal valves. Water of a desired salt concentration could be added through these valves at any convenient time during the progress of the test. The amount of water made available to the sample was measured by burettes with a capacity of 25 milliliters.

The major portion of this research project was directed toward the measurement of pore-water pressures with controlled changes in the chemical environment.

To enable these pore pressures to be measured, a special pore pressure measuring device in the form of a pressure transducer was incorporated into the design of the consolidation cell. The model and type of pressure

transducer employed were chosen on the basis of anticipated pore pressures to be measured. In conjunction with a Baldwin type strain gauge indicator, pore pressures were recorded. This laboratory equipment will be discussed fully in section 2.4 of this chapter.

2.4 Laboratory Equipment Employed

Much of the equipment employed in this research project is to be found in any standard soil testing laboratory. The following sections will therefore describe only that equipment which had to be developed for this project. A description of the standard laboratory consolidation apparatus is necessary for a discussion of mechanical inaccuracies.

(1) Special Consolidation Cell

At the outset of this research program there appeared to be no conventional apparatus in Canada which could be used for the control of leaching fluid and the measurement of pore-water pressures in a one-dimensional consolidation test. Therefore, suitable equipment had to be designed and constructed to accomplish the desired ends.

The initial design of the consolidation cell was suggested by Hardy (1964). The author investigated

the proposed design and concluded that it would be able to measure and control the desired variables. A preliminary set of machine drawings were prepared and submitted to a machinist for construction. The various components of the cell are shown in an assemble drawing in FIGURE 1.

With the exception of the rubber o-rings, the complete cell was constructed of stainless steel. This was necessary to eliminate the possibility of electrolytic action between the cell and the salt solution.

Briefly, a list of the component parts and their functions is given below:

1. Upper and Lower Soil Chambers

These chambers comprise the bulk of the consolidation cell. Soil samples were placed in these chambers for testing purposes. When the cell is assembled, the upper and lower chambers are separated only by the center porous plate and two filter papers.

2. Upper and Lower Pistons

The upper and lower pistons were designed to transmit load to the upper and lower soil

samples and to allow drainage from these samples. When inserted into the consolidation cell, they serve as a loading device and as a pressure seal.

3. Connecting Bolts

The connecting bolts serve to connect the upper and lower soil chambers into one continuous unit.

4. Stainless Steel Porous Plates

Three stainless steel porous plates were used in this cell. The stainless steel plates were desired in place of the usual brass or stone plates for two reasons; they are very much stronger, and they serve to eliminate the possibility of electrolytic action between the porous plates and the salt solution.

5. Circle Seal Valves

The design of the cell included six, zero-volume-change, Circle Seal Valves. These valves allowed drainage into and out of the upper and lower soil chambers. One of the valves allowed the area directly in front of the pressure transducer to be de-aired.

6. Rubber o-rings

The function of three rubber o-rings was to serve as a pressure seal between the upper and lower soil chambers, and between the upper and lower soil sample and the atmosphere.

For test purposes, the special consolidation cell was used in the Wykeham-Farrance consolidation apparatus. PLATES 1 & 2 show the special consolidation cell as it was used in conjunction with this apparatus.

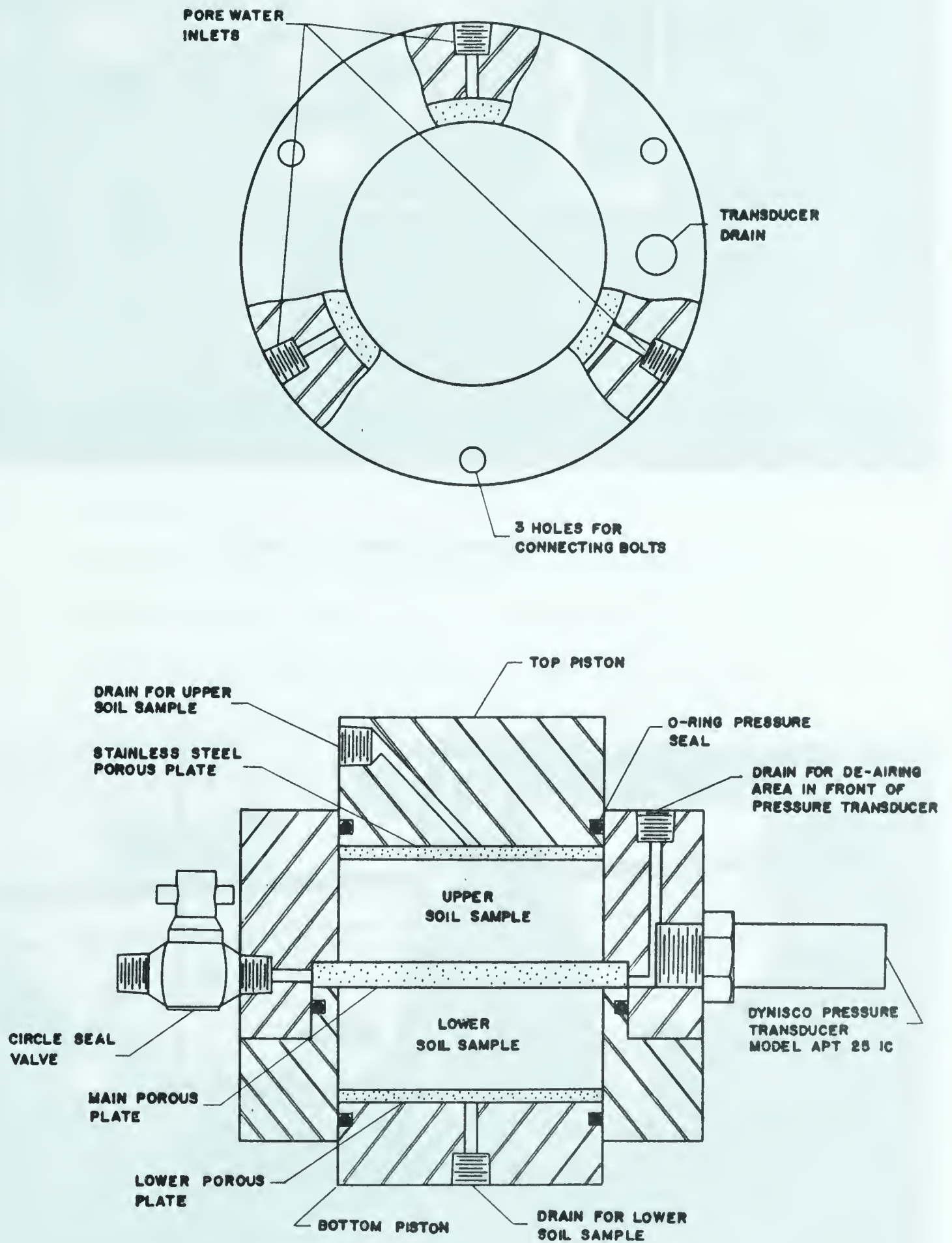


FIGURE I CONSOLIDATION CELL FOR SWELLING
PRESSURE STUDY

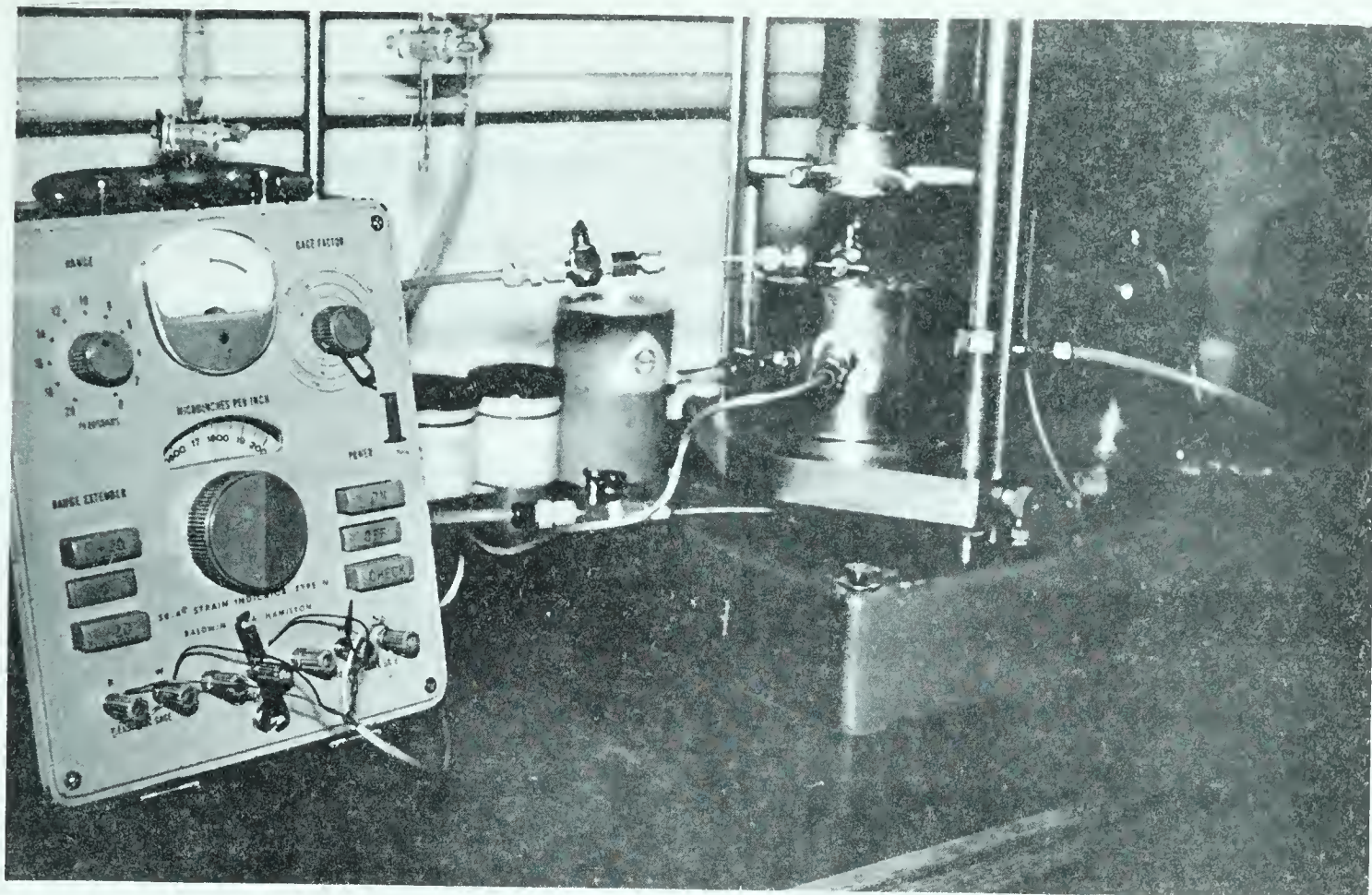


Plate 1. Special Consolidation Cell

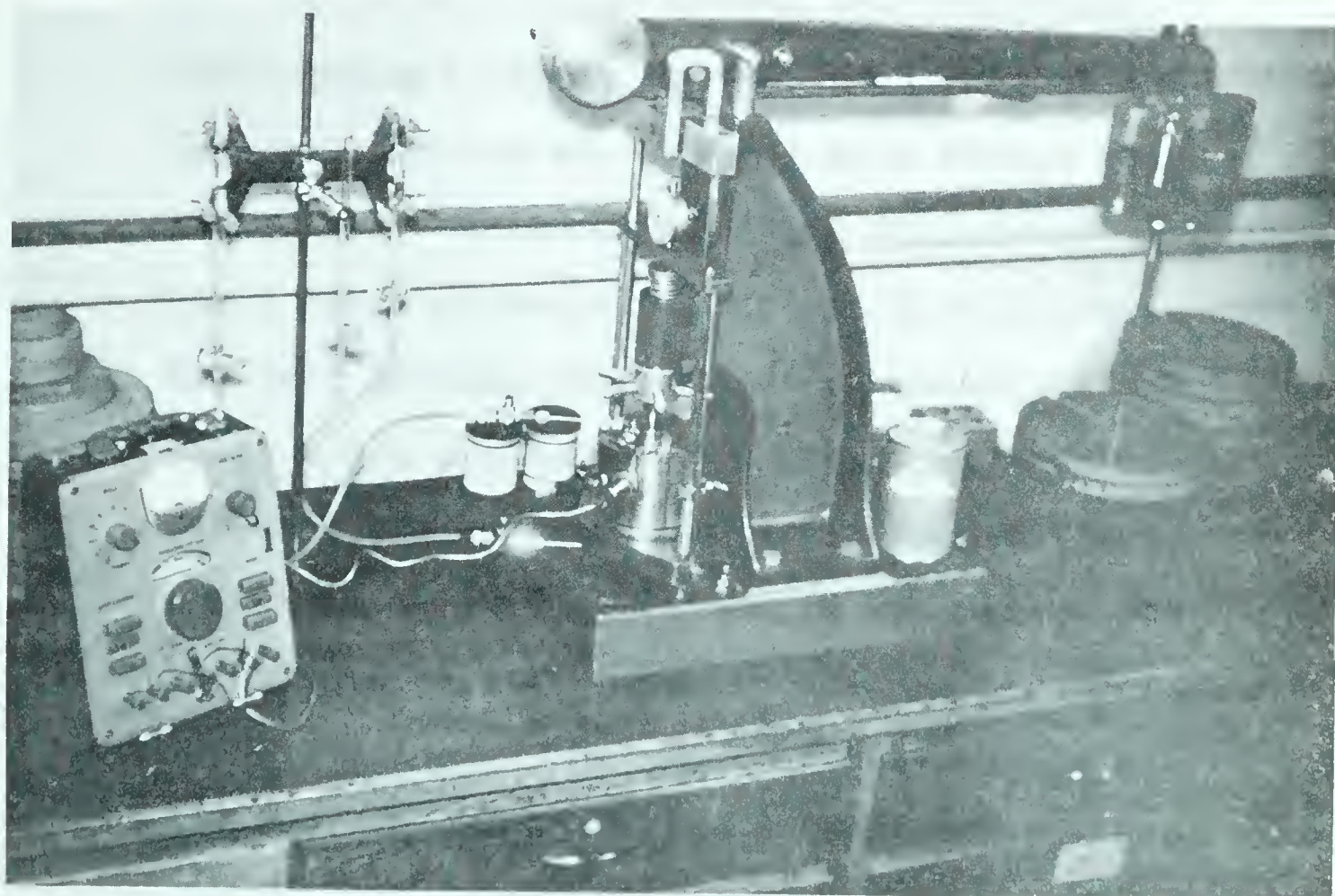


Plate 2. Wykeham - Farrance Consolidation Apparatus



1877



1877

(2) Pressure Transducer

In a previous section of this chapter (section 2.3) it was mentioned that a pressure transducer was employed to measure pore-water pressures. It is desirable to describe this apparatus more fully.

The model of pressure transducer employed was a Dynisco APT 25-1C. This transducer has a measuring range of 0 - 100 psi positive pressure. The range for negative pressure is known only to zero gauge.

The principle behind the pressure transducer is a strain gauge mechanism. As fluid pressure is applied to the diaphragm, a strain occurs in the wheatstone bridge within the transducer. This causes a change in the resistance of the bridge which can be measured by a strain gauge indicator. Calibration of the transducer against known fluid pressure results in a calibration curve for both loading and unloading conditions. The calibration curves for the transducer employed in this program are included in Appendix C to this thesis. Also included in Appendix C are detailed specifications for the pressure transducer. Although the range of pressure is known only to zero gauge, pressures below this level can also be measured. It is necessary

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to assume that the calibration curve below zero gauge is of the same characteristic shape as that above zero.

(3) Standard Laboratory Consolidation Apparatus

In addition to constant-volume swell tests performed in the special consolidation cell, free-swell tests and constant-volume swell tests were also performed in standard laboratory consolidometers. The type of consolidometers used for these latter tests was the Wykeham-Farrance Bench model. These consolidometers are equipped with stainless steel, fixed rings and a British dial indicator accurate to 0.0001 inch. The method of applying a load to the soil samples is through a lever system that is listed as having a multiplication factor of 11:1.

In previous research at the University of Alberta, Fredlund (1964) indicated that the multiplication factor of the Wykeham-Farrance apparatus may vary as much as $\pm 1\frac{1}{2}\%$. Fredlund also conducted a study on the compressibility of the consolidation apparatus and found it to significantly affect test results. Since the soil used in the present research was highly overconsolidated, very little volume change was

anticipated. It was therefore imperative that the mechanical inaccuracies of the equipment employed be taken into consideration.

The results of Fredlund's study were applied as corrections to the standard laboratory swell tests performed by the author. Calibration curves for the apparatus compressibility are included in Appendix C of this thesis.

2.5 General Testing Procedures

The following sections outline the general test procedures employed in this study. Whenever possible standard laboratory procedures were followed.

This section also includes procedures employed for sample preparation, duplication of the soluble pore-water salts, calibration of the pressure transducer, and compressibility of the testing apparatus. Most of the information gathered from these latter laboratory procedures is given in the appendices of this thesis.

(1) Sample Preparation

The soil specimens for both the standard laboratory swell tests and those performed in the special consolidation cell were prepared in a similar manner. Undisturbed soil samples from the same bore hole at a depth of 108.6 - 109.1 feet were obtained from the Prairie Farm Rehabilitation Administration (P.F.R.A.). These

were originally sealed with a paraffin coating so as not to lose moisture.

The soil samples were trimmed to the size of the soil chamber for the special consolidation tests and to the size of the Wykeham-Farrance rings for the standard laboratory tests. After trimming, the samples were placed in a plastic bag and re-sealed with a paraffin coating until ready for testing.

(2) Calibration of the Pressure Transducer

The calibration of the pressure transducer was carried out in the Wykeham-Farrance consolidation apparatus. It was thought necessary to calibrate the transducer in this equipment since any mechanical inaccuracies would be cancelled out.

A Baldwin strain gauge indicator was used in conjunction with the transducer. This type of indicator records the amount of strain taking place within the transducer in micro-inches per inch.

The first step in the calibration was to obtain a strain reading under atmospheric pressure. The transducer was then inserted into the consolidation cell and de-aired, distilled water was added. The complete system was then de-aired

by flushing water through the cell. The valves were then closed and a strain reading under the weight of the top piston was obtained. Subsequent loads were added to the cell in approximately double increments up to a maximum of 6.5 kg/cm^2 (93.0 psi). Strain readings were taken immediately after each load was applied. The loads were then removed in the same increments and strain readings were taken immediately after each load removal.

The complete cycle of loading and unloading was conducted four separate times. Calibration curves for the loading and unloading cycles were constructed on the basis of the average of these readings.

(3) Piston Friction in The Consolidation Apparatus

An attempt was made to measure the amount of friction between the upper and lower pistons and the side walls of the consolidation cell. The clearance between the pistons and the side walls without the O-rings was 0.018 inches. This amount of clearance allowed the pistons to move up and down in the cell without frictional resistance. However, to sustain an effective pressure seal, it was necessary to use the O-rings.

The first part of the paper is devoted to a discussion of the general principles of the theory of the structure of the atom. It is shown that the structure of the atom is determined by the laws of quantum mechanics, which are based on the principle of the conservation of energy and the principle of the conservation of momentum. The second part of the paper is devoted to a discussion of the experimental results obtained in the study of the structure of the atom. It is shown that the experimental results are in good agreement with the theoretical predictions of quantum mechanics. The third part of the paper is devoted to a discussion of the applications of the theory of the structure of the atom. It is shown that the theory of the structure of the atom has many important applications in the fields of physics, chemistry, and biology.

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When they were present, some frictional resistance was also present and had to be taken into consideration.

A series of load-friction tests was conducted by placing the pistons inside the cell and measuring the load necessary to cause their movement. The walls of the consolidation cell were initially covered with silicon grease. The cell was then assembled and placed into the Wykeham-Farrance consolidation apparatus. Small loads were applied to the loading pan and the amount of load necessary to cause movement was recorded.

Thirty separate determinations of this load were conducted. Values were recorded to the nearest 5 grams. The average value of these determinations was 150 grams on the loading pan. The standard deviation of these determinations was ± 10 grams. To correct the loads applied to the samples during both special consolidation tests, a value of 150 grams was subtracted from the load on the pan.

(4) Duplication of the Pore-water Salts

To control the chemical environment of the soil-water system, the soluble salts in the

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pore water were analyzed in the laboratory.

These salts were determined using the following laboratory methods (APHA, 1955).

- | | |
|----------------------|--|
| 1. Calcium content; | Verzinate method
(E.D.T.A.) |
| 2. Sulphate content; | Precipitation with
barium chloride and
by the use of a spectro-
photometer. |
| 3. Chloride; | Silver nitrate titration |
| 4. Magnesium; | E.D.T.A. method |
| 5. Sodium; | by difference. |

The results of this analysis are presented in TABLE II.

In order to duplicate the salt concentration in the pore water, various combinations of four salt compounds were used. These include:

Calcium Chloride (CaCl_2)

Magnesium Sulphate (MgSO_4)

Sodium Sulphate (Na_2SO_4)

Sodium Bi-Carbonate (NaHCO_3)

The amount of each compound necessary for exact duplication is presented in TABLE III.

(5) Compressibility of the Testing Apparatus

The results of previous studies (Fredlund, 1964) on the compressibility of the Wykeham-Farrance testing apparatus indicated to the

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author that a similar study was desirable for the special consolidation cell. The compressibility of the apparatus was determined by using the upper and lower pistons, the stainless steel porous plates, and four (4) No. 54 Whatman filter papers. These items were placed in the Wykeham-Farrance Bench Model No. 3 without lateral support. A small load was applied and changes in the dial indicator were recorded at 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, 30.0, and 60.0 minutes. Immediately following the 60 minute reading, an additional load was applied and the cycle was repeated. Loading continued in this manner up to a maximum of 26.70 kg/cm^2 .

The application of a particular load for a set time interval allowed a family of time-compressibility-pressure curves to be drawn. From these curves, a net compressibility-pressure curve was constructed for a 24 hour time period. The family of time-compressibility-pressure curves and the net compressibility-pressure curve are presented in Appendix C of this thesis.

(6) Special Consolidation Tests

Preliminary to testing, a solution containing the desired pore water salt concentration was prepared. The salt concentration duplicated that determined in the laboratory, with the exception of the sodium concentration.

The concentration of the sodium salts for both tests was 0.3 normal. The remainder of the soluble pore-water salts, calcium and magnesium, was as in the original pore-water salt analysis.

Before the swelling test could begin, it was necessary to de-air the stainless steel porous plates. This was accomplished by boiling them in a solution containing the concentration of salts desired. After boiling for a period of 30 minutes, de-aired, distilled, water was added to the beaker to account for losses of the salt solution through boiling.

The complete cell, including the burettes and the nylon lines connecting the burettes to the cell, were washed in a soap solution and rinsed with distilled water. The cell was then assembled and the burettes filled with the salt solution.

The soil samples were then taken from the

dial indicator. Each time a change of between 0.0001 - 0.0002 inches took place in the dial indicator, a small load was added to the soil. The small loads never compressed the soil more than 0.0003 inches during one loading period. Initially, it was necessary to add loads at one minute intervals. These time intervals increased geometrically until the sample no longer had a tendency for swell. At this time it was believed that an equilibrium condition had been attained in which the external stresses acting on the sample were just counter-balanced by the tendency of the soil to swell.

At this point in the progress of the test, the center porous plate was flushed with de-aired, distilled water. This reduced the concentration of the salts in the porous plate from 0.3 normal sodium to zero. The soil thus had access to a solution of much lower salt concentration than was formerly present in the pore water.

The leaching process was accomplished by connecting the center burette line to a reservoir. Under a small pressure (approximately 2.5 psi) the water flowed through the center

porous plate and into a container. This leaching process was continued for a period of approximately 30 minutes to ensure that the salt solution originally in the center porous plate had been displaced with distilled water.

Upon completion of the leaching process, all valves to the cell were closed. Readings from the strain gauge indicator and from the vertical dial indicator were taken. However, the sample started to swell during the leaching period. This was apparent from changes in the vertical dial indicator. Additional loads were added to the sample in order to keep swell to a minimum. With the addition of each load, pressure transducer readings were taken from the strain gauge indicator in order to assess changes which might have occurred in the pore-water pressure.

The tendency for swell after the leaching process had taken place was considerably reduced from that present before leaching. However, it was evident that this tendency was present. Readings from the strain gauge indicator showed the pore-water pressure to be decreasing below the original value measured

just after leaching had taken place. The decrease was, however, very slight (FIGURES 12, 13).

Additional loads were applied to the soil whenever the vertical dial indicator had changed 0.0001 inch. These additional loads were applied until it was evident that the tendency for swell had become negligible. For both tests, the increase in load above that before leaching took place was approximately 1.0 kg/cm^2 .

At this point, it was assumed that an equilibrium condition had again been reached. Thus, the total load acting on the sample was equal to the swelling forces developed in the soil. Actually the equilibrium point was finally exceeded because the last small load increment caused a measureable amount of consolidation. In test number 1, the sample was allowed to remain under this environment for a period of 44 hours. No significant volume change occurred during this time interval. Test number 2 was terminated approximately 26 hours after the last small load increment was applied.

A continuation of the test into the consolidation phase was carried out only in test

number 1. This was accomplished by opening the valves to the top and bottom pistons and proceeding through the loading and unloading cycles. When the rebound portion of the consolidation test was complete, moisture contents were determined from the sample.

Before test number 2 was terminated, a pore pressure reaction test was conducted. This test was accomplished by loading the soil increments with no drainage allowed from the sample. Loads of 0.95, 1.89, 1.89 and 1.89 kg/cm² were applied to the soil for a time period of 15, 30, and 60 minutes per load. Strain gauge indicator readings were taken at intervals of 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, 30.0, and 60.0 minutes. The results of this pore pressure reaction test are given in FIGURE D1 in Appendix D.

The pore pressure reaction test indicated a very low value of the parameter B. For this thesis, the definition of B is the ratio of the pore-water pressure measured to the total stress applied to the sample during the reaction test. After 15 minutes, the B value was approximately 0.6 for each load increment.

(7) Standard Laboratory Swell Tests

In addition to the constant-volume swell tests performed in the special consolidation cell, a series of free-swell and constant-volume swell tests were conducted in standard laboratory testing apparatus. The equipment used in these test was the Wykeham-Farrance Bench model.

Both constant-volume and free-swell tests were carried out using various salt solutions as the immersing fluid. These tests were performed to evaluate the swelling properties of the soil under controlled chemical environment. Solutions containing salt concentrations of 0.0 (distilled water), 0.2, 0.4, and 0.6 normal sodium chloride were used as the immersing fluid for both types of swell tests.

The test procedures for constant-volume and free-swell test performed in consolidation equipment are the same as that outlined by Lambe (1958) for consolidation of soils. The following exceptions apply to procedures followed in this program.

1. Free swell was allowed the samples under an initially small load. The time allowed

for free swell varied from 2300 hours in the case of the sample using distilled water as the immersing fluid to 19 hours for immersing fluids of greater salt concentrations. Once free swell was complete, the consolidation cycle of loading was followed.

2. In the constant-volume swell tests, small increments of load were added as the soil indicated a tendency for swell. When this tendency was no longer apparent, the consolidation cycle of loading and unloading was followed.

2.6 Summary

This chapter outlines the experimental procedure employed to investigate physico-chemical effects in a swelling soil. Special laboratory equipment was necessary to measure variables during the tests performed. This equipment included a newly designed consolidation cell which employed a pressure transducer. Standard laboratory equipment was also used.

Standard laboratory tests followed the procedures outlined by ASTM (1958) and Lambe (1958). For those tests performed in the special consolidation cell, new procedures were developed.

CHAPTER III

PRESENTATION OF TEST RESULTS

3.1 Introduction

The object of this study was to investigate the influence of changes in the physico-chemical environment on the stresses developed during a one-dimensional consolidation test. The results of this investigation may be presented as changes in the total stress and in the pore-water pressure with changes in the physico-chemical environment. In this manner, changes in the effective stresses may be analyzed.

The values for total stresses and pore-water pressures are given in kg/cm^2 . The terms concentration, salt content, and electrolyte concentration refer to the concentration of the pore water salts expressed in normality.

The study also included an investigation into the swelling properties for a highly-overconsolidated clay soil. The results of this study are presented with the aid of dial-deflection vs. log-pressure curves.

3.2 Soil Classification Tests

TABLE I summarizes the classification tests performed on Bearpaw Shale. The physical classification tests include specific gravity, Atterberg limits, and grain size analysis (hydrometer method). The mineralogy and physico-chemical classification tests include the type of clay mineral present and the type and amount of exchangeable cation present.

The values of specific gravity range from 2.72 to 2.75 with an average value of 2.74. The Atterberg limits range from 103.8% for the liquid limit to 19.8% for the shrinkage limit. The plasticity index was 69.3% indicating an inorganic soil of high plasticity. The grain size analysis shows that the soil was composed mainly of clay-sized particles less than 0.002mm in diameter. The activity of the soil is 1.19.

The mineralogical composition of the soil was determined by X-ray diffraction methods. The predominant clay mineral was montmorillonite. This mineral constitutes about 80% of the clay mineral faction. The remaining clay mineral constituents were kaolinite and chlorite.

The amount of exchangeable sodium, calcium, magnesium, and potassium was determined by standard methods at the Soil Science Department, University of Alberta.

The predominant exchange cation was sodium with a value of 24.4 milliequivalents per 100 grams air-dry soil (me/100 ads). Other exchange cations present include calcium (19.5 me/100 ads), magnesium (3.3 me/100 ads), and potassium (1.0 me/100 ads). The quantity of exchangeable cations includes adsorbed cations as well as free cations in the pore water.

The total exchange capacity (TEC) was also determined by the Soil Science Department. For the soil sample involved, the TEC was 42.8 me/100 ads. This value is a measure of the adsorbed cations only. The difference between the sum of the exchangeable cations and the TEC is a measure of the free electrolyte in the pore water.

3.3 Chemical Characteristics of the Pore Water

TABLE II lists the results of a laboratory analysis of the chemical constituents contained in the pore water of Bearpaw Shale. Included in this analysis were tests for Conductivity, Total Hardness, Alkalinity, and Salt Ions Present. These tests were conducted in accordance with standard methods for the examination of water (APHA, 1955).

The results of these tests indicate that the pore water of Bearpaw Shale contains a predominant amount of free sodium ions. The amount present was 3,320

parts per million (PPM). Other soluble salts present include calcium (85PPM), and magnesium (17PPM).

In TABLE III is presented a list of compounds used to duplicate the soluble pore water salts. The compounds used were standard laboratory salts. The amounts necessary are given for reference for future research.

TABLE I

SUMMARY OF CLASSIFICATION TESTS ON BEARPAW SHALE

TEST	RESULT
SPECIFIC GRAVITY*	2.74
ATTERBERG LIMITS	
LIQUID LIMIT	103.8
PLASTIC LIMIT	34.5
SHRINKAGE LIMIT	19.8
PLASTICITY INDEX	69.3
GRAIN SIZE ANALYSIS**	
% SAND SIZES	4.0
% SILT SIZES	37.0
% CLAY SIZES	59.0
ACTIVITY	1.19
CLAY MINERALOGY	
MONTMORILLONITE	80
KAOLINITE	—
CHLORITE	—
EXCHANGE CAPACITY***	42.8
EXCHANGEABLE CATIONS***	
SODIUM	24.4
CALCIUM	19.5
MAGNESIUM	3.3
POTASSIUM	1.0

* AVERAGE OF THREE TESTS

** M.I.T. GRAIN SIZE SCALE

*** MILLIEQUIVALENTS PER 100 GRAMS AIR DRY SOIL

TABLE II

PORE WATER CHEMICAL CHARACTERISTICS

CHEMICAL CHARACTERISTIC	CONCENTRATION VALUE
CONDUCTIVITY	
MEASURED	850 μ MOHS
COMPUTED	857 μ MOHS
TOTAL HARDNESS	
SAMPLE	16 PPM*
CALCIUM	12 PPM
ALKALINITY	
AS CaCO_3	350 PPM [ALL HCO_3]
SO_4 AS SO_4	75 PPM
CL AS CL	10 PPM
SALT ION PRESENT	
CA	85 PPM
MG	17 PPM
NA	3320 PPM
HCO_3	7250 PPM
SO_4	1290 PPM
CL	170 PPM

*PARTS PER MILLION

TABLE III

DUPLICATION OF SOLUBLE PORE-WATER SALTS

SALT COMPOUND	EPM ✱	EQUIVALENT WEIGHT	GRAMS/LITER	NORMALITY
CaCl_2	4.2	55.00	0.2265	0.0042
MgSO_4	1.4	60.18	0.0852	0.0014
Na_2SO_4	25.0	71.03	1.775	0.025
NaHCO_3	119	84.00	10.00	0.119

✱ EQUIVALENT PARTS PER MILLION

TABLE I

Summary of the results of the experiments

Experiment	Time	Results		Remarks
		Value	Unit	
1	100	1.2	g	1.2
2	200	1.5	g	1.5
3	300	1.8	g	1.8
4	400	2.0	g	2.0

Source: [illegible]

3.4 Standard Laboratory Swell Tests

The results of the standard laboratory free-swell and constant-volume tests are listed in TABLE IV and are presented graphically in FIGURES 2 through 11.

The soil samples involved in these tests were undisturbed. They were carved from a chunk sample obtained from the same bore hole at a depth of 108.6 - 109.1 feet.

Moisture content determinations were made from all soil samples. These moisture contents are listed in TABLE IV. It is noted that they vary from 23.01% to 25.33%. This is a variation of 9.2%. All samples had an initial moisture content greater than the shrinkage limit (19.2%). Therefore, they should have been saturated.

The initial degree of saturation was calculated from the dimensions of the consolidation ring and from the dry weight of the soil samples after the tests were complete. For all samples tested, the degree of saturation did not vary more than 7% from complete saturation (100%). The fact that the calculated degree of saturation was greater than 100% is believed due to experimental accuracy in measuring the specific gravity.

Since the soil samples came from the same depth, the

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REIGN OF KING CHARLES THE FIRST

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over-burden pressure acting on the sample before removal from the ground should have been identical. Once removed from the in-place conditions, each sample is assumed to be subject to capillary forces of the same magnitude. When the samples were immersed in a water solution, the capillary forces are destroyed and swelling occurs. However, for different electrolyte solutions, differences in both the amount of swell as well as the swelling pressure were noted.

The graphical results of the laboratory swell tests are presented in the form of deflection vs. log-pressure curves. These curves have been corrected for compressibility of the apparatus and of the filter paper used in the consolidometers. The swelling pressures for both the constant-volume and free-swell tests have been defined as the pressure at which the consolidation curve crosses the zero-dial-deflection line (FIGURE 2). In the constant-volume tests presented here, this definition of swelling pressure was necessary because some volume change took place during the initial loading period. With the laboratory equipment employed, it was not possible to add load continuously as the sample gradually swelled. Rather, it was necessary to allow a small increase before the addition of each finite load.

FIGURES 2, 3, and 4 show the consolidation curves for free-swell and constant-volume tests in which solu-

tions of varying concentrations of sodium chloride were used as the immersing fluid. The conditions under which each of these swell tests was conducted are listed in TABLE IV.

FIGURE 2 presents the consolidation curves for swell tests using distilled water as the immersing fluid. For the free-swell test, the sample was allowed to swell for 2,329 hours under a small confining load of 0.185 kg/cm^2 . An increase in height of 6.44% of the original height of the sample was recorded during this period. The swelling pressure for this test was found to be 11.7 kg/cm^2 .

For the constant-volume swell test, the swelling pressure was 11.7 kg/cm^2 . It is noted that these two swelling pressures are identical so far as graphical interpretation allows.

The consolidation curves for swell tests in which a solution containing 0.2 normal sodium chloride was used as the immersing fluid are given in FIGURE 3. The initial confining pressure for the free-swell test was 0.194 kg/cm^2 . Under this small load, the sample was allowed to swell for 72 hours. The increase in height above its original height was 3.26%. The swelling pressure was 6.2 kg/cm^2 . This is approximately one-half of that obtained from the sample immersed in distilled water. The swelling pressure for the constant-volume

test was 11.5 kg/cm^2 . It is noted that no significant change occurred in this swelling pressure over that determined from the sample immersed in distilled water.

FIGURES 4 and 5 present the results of swell tests for immersing solutions of 0.4 and 0.6 normal sodium chloride respectively. In the free-swell tests, the confining pressures were the same (0.184 Kg/cm^2). For the sample immersed in 0.4 normal sodium chloride, an increase in height of 2.95% was recorded over a period of 100 hours free swell. The swelling pressure for this sample was 4.2 kg/cm^2 . For the sample immersed in 0.6 normal sodium chloride, an increase in height of 2.33% was recorded. This sample required only 19.5 hours to complete the free swell. The swelling pressure was 4.3 kg/cm^2 , an increase of 0.1 kg/cm^2 over the previous sample. Both free-swell swelling pressures were lower than those for the samples immersed in distilled water and in 0.2 normal sodium chloride.

The constant-volume tests for 0.4 and 0.6 normal sodium chloride immersing solutions showed much the same trend. Swelling pressures of both samples decreased over the previous two constant-volume tests. The swelling pressure for the sample immersed in 0.4 normal sodium chloride was 8.0 kg/cm^2 ; that for the sample immersed in 0.6 normal sodium chloride was 7.4 kg/cm^2 .

A graphical summary of swelling pressure information from the free-swell and constant-volume swell tests is presented in FIGURE 6. In this FIGURE, swelling pressure was plotted as ordinate against salt concentration of the immersing fluid as abscissa. Both types of tests are included.

The relationships illustrated by FIGURE 6 are represented by dashed lines since the number of points are considered inadequate to express the variation in swelling pressure for all values of immersing concentration. However, the trend is definite. It appears that for the sample involved, free-swell swelling pressures decrease with increased concentration of the immersing fluid. The constant-volume swelling pressure also shows a downward trend as the concentration of the immersing solution increases.

FIGURES 7, 8, 9, and 10, present the relationships between free swell and time. These relationships were obtained from the results of the free-swell tests in which varying concentrations of sodium chloride were used as the immersing solution. The amount of swell, expressed as a percentage of the samples original height, is plotted as ordinate. Time, expressed in hours is plotted as abscissa.

FIGURE 7 shows the free-swell-time curve for the sample immersed in distilled water. This sample was

allowed to swell for 2,329 hours. The test was terminated because of a time limitation.

It is noted that the desire for water had not been satisfied after this time period. The curve rises slowly in the initial stages of the test and then becomes approximately a straight line. After 1000 hours (approximately 100 days) the curve levels off somewhat.

FIGURE 8 presents the free-swell vs. time curve for the sample immersed in a 0.2 normal solution of sodium chloride. For this sample, the desire for water was effectively satisfied after 100 hours of free swell. The shape of the curve is characteristic of the swelling process and is the mirror image of a consolidation-time curve.

FIGURES 9 and 10 present the free swell-time curves for samples immersed in solutions of 0.4 and 0.6 normal sodium chloride respectively. For the sample immersed in 0.4 normal sodium chloride, the desire for water was not completely satisfied. After 72 hours, the curve had leveled off somewhat, but was still increasing at a significant rate. However, extrapolation of this curve out to the 200 hour time point results in an increase of swell of only 0.10%.

For the sample immersed in 0.6 normal sodium chloride, the desire for water was satisfied in a very short time period. Only 19.5 hours were required to bring the sample

to equilibrium with respect to swelling potential.

A graphical summary of the amount of free swell occurring in the free-swell tests is presented in FIGURE 11. Free swell, expressed as a percentage of the samples original height, was plotted as ordinate. The sodium chloride concentration of the immersing fluid was plotted as abscissa. Once again, the proposed relationship is illustrated with a dashed line. The data are inadequate to define the variation in free swell for all values of immersing concentration.

It is noted that the amount of free swell decreases abruptly with increase in electrolyte concentration. The difference between the amount of free swell obtained under distilled water conditions and that obtained under a 0.2 normal sodium chloride condition is approximately 50%. As the concentration of the immersing fluid increases, the amount of free swell gradually decreases. Thus, it appears that, for the soil samples tested, an electrolyte solution does have an effect on the amount of swell.

TABLE IV
SUMMARY OF TEST RESULTS, STANDARD LABORATORY SWELL TESTS

TEST TYPE	SALT CONCENTRATION OF IMMERSING FLUID [NACL] NORMALITY	INITIAL CONFINING PRESSURE $\frac{kg}{cm^2}$	MAXIMUM CONSOLIDATION PRESSURE $\frac{kg}{cm^2}$	FINAL REBOUND PRESSURE $\frac{kg}{cm^2}$	SWELL, PERCENT OF ORIGINAL HEIGHT	SWELLING PRESSURE $\frac{kg}{cm^2}$	MOISTURE CONTENT		VOID RATIO		INITIAL DEGREE OF SATURATION %
							INITIAL %	FINAL %	INITIAL	FINAL	
FREE SWELL	0.0	0.185	23.02	0.220	6.44	11.7	24.23	25.22	0.59	0.69	102.52
FREE SWELL	0.2	0.194	24.20	0.194	3.26	6.2	23.99	25.47	0.65	0.70	101.65
FREE SWELL	0.4	0.184	20.70	0.184	2.95	4.2	23.63	24.47	0.62	0.67	104.31
FREE SWELL	0.6	0.184	15.12	0.184	2.33	4.3	23.01	23.73	0.57	0.65	101.43
CONSTANT VOLUME	0.0	0.01	24.38	0.46	-----	11.7	24.41	25.70	0.65	0.70	103.55
CONSTANT VOLUME	0.2	0.01	23.78	0.59	-----	11.5	23.82	25.32	0.65	0.69	99.83
CONSTANT VOLUME	0.4	0.01	25.70	0.134	-----	8.0	25.33	25.99	0.66	0.71	106.57
CONSTANT VOLUME	0.6	0.01	24.72	0.177	-----	7.4	22.53	24.13	0.65	0.66	95.40

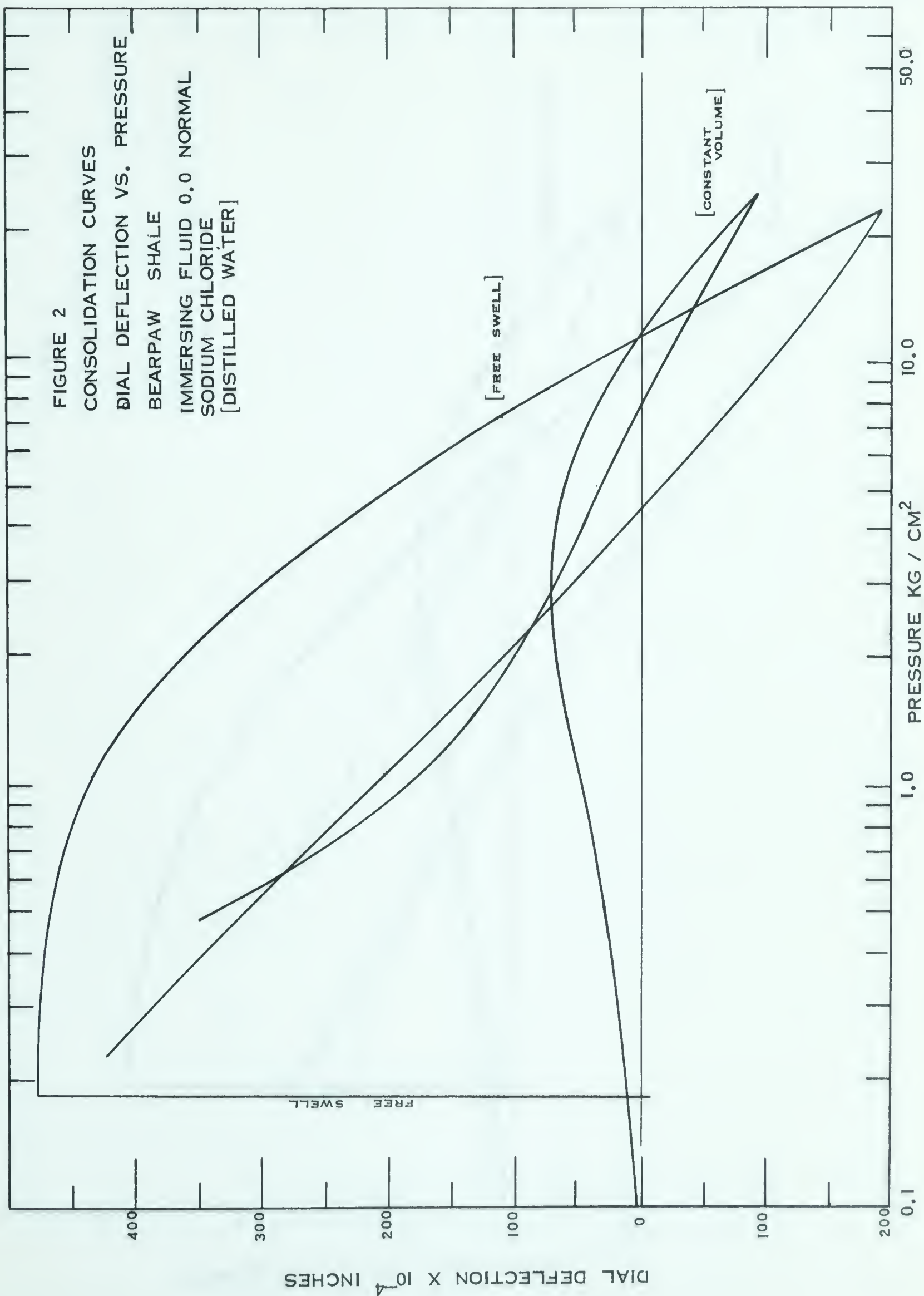
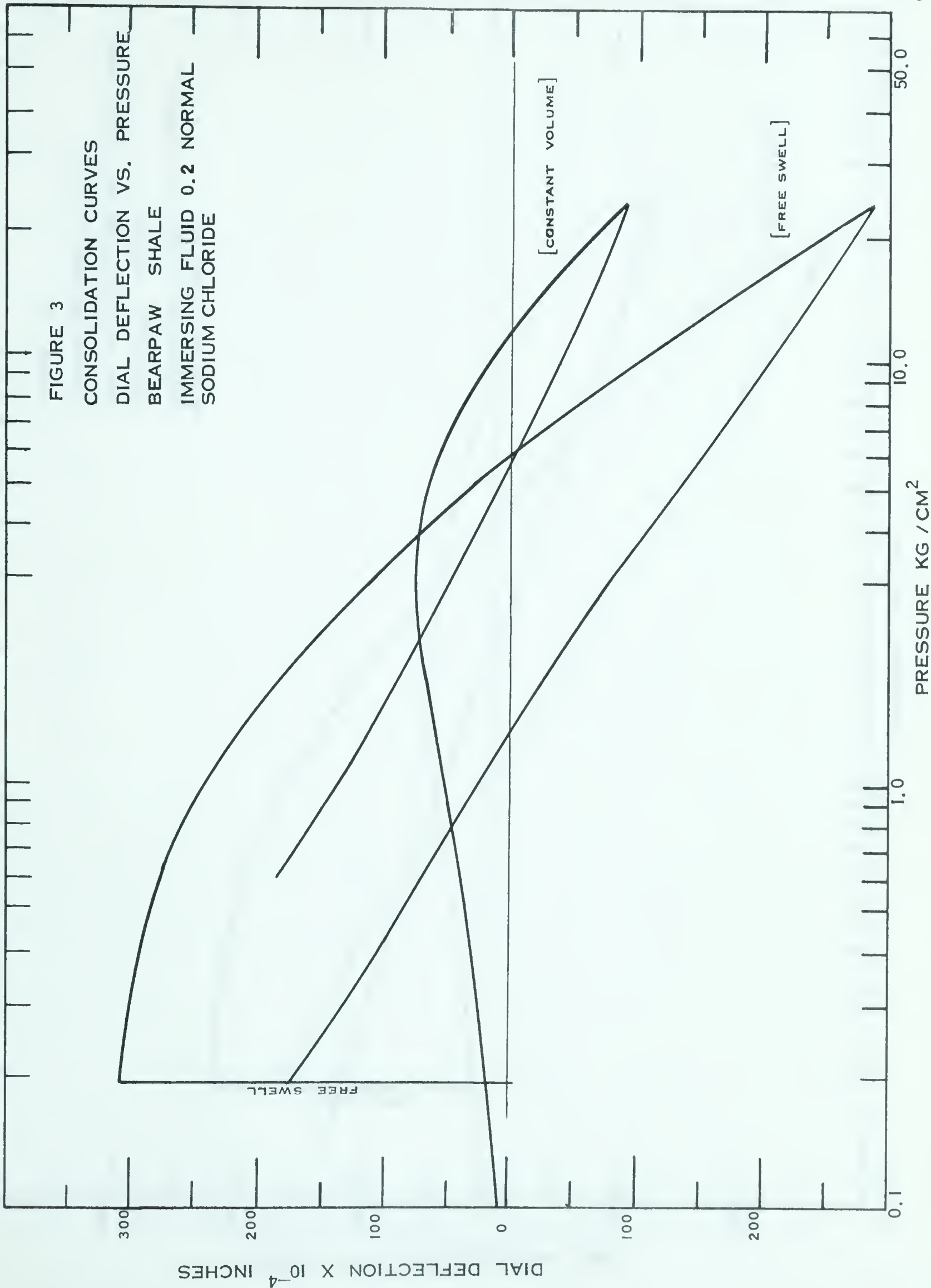


FIGURE 3
CONSOLIDATION CURVES
DIAL DEFLECTION VS. PRESSURE
BEARPAW SHALE
IMMERSING FLUID 0.2 NORMAL
SODIUM CHLORIDE



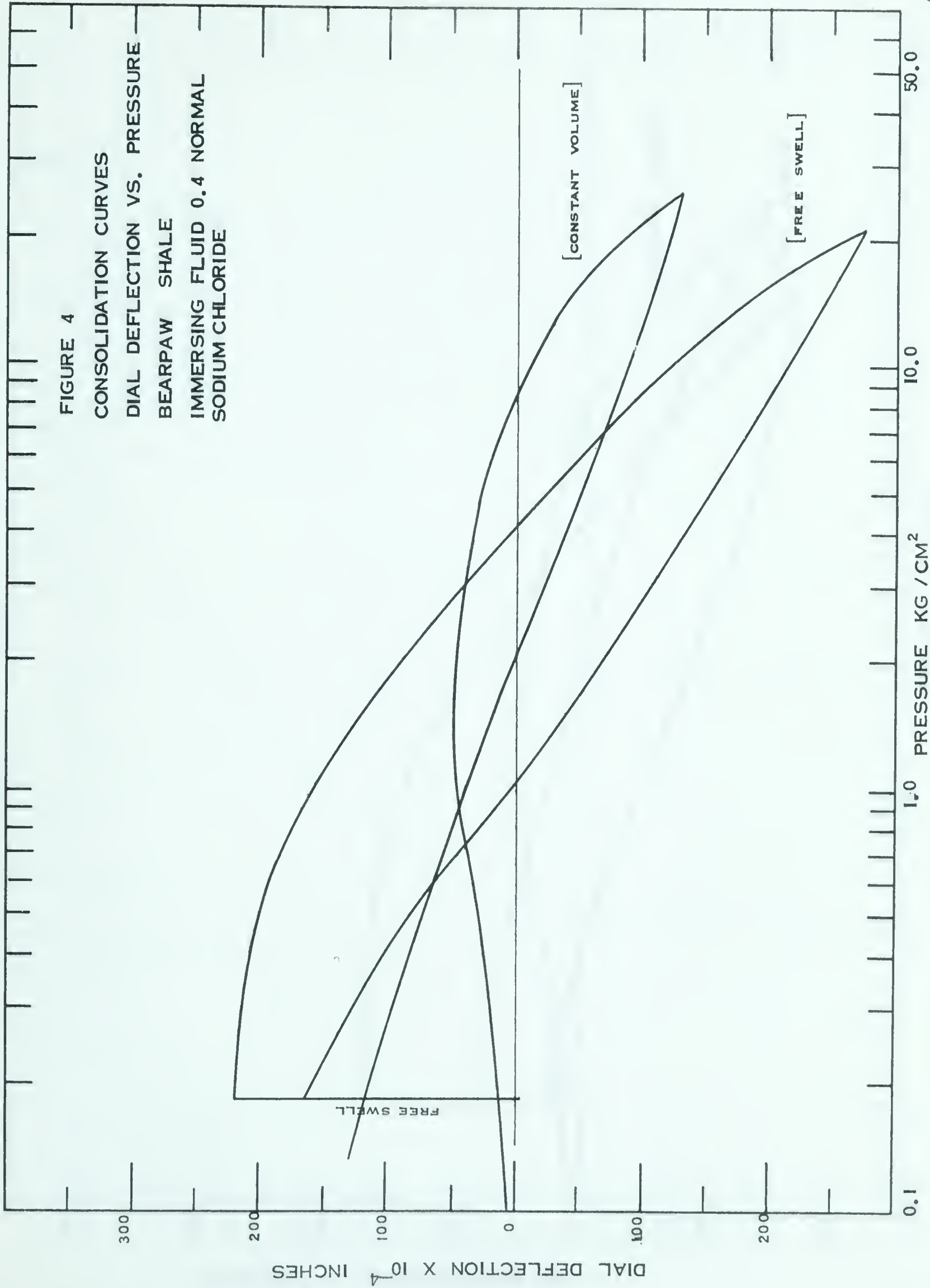


FIGURE 4
CONSOLIDATION CURVES
DIAL DEFLECTION VS. PRESSURE
BEARPAW SHALE
IMMERSING FLUID 0.4 NORMAL
SODIUM CHLORIDE

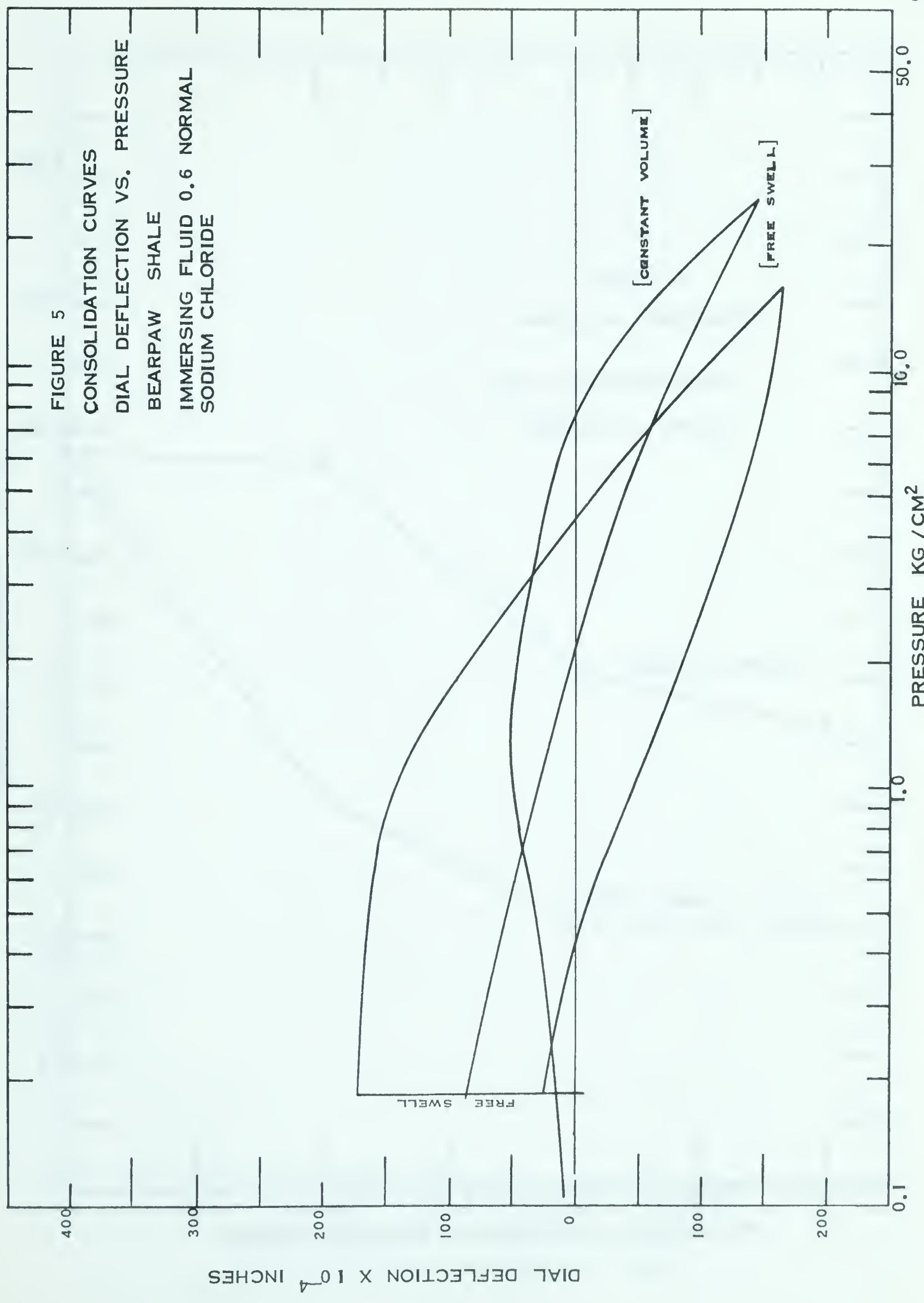
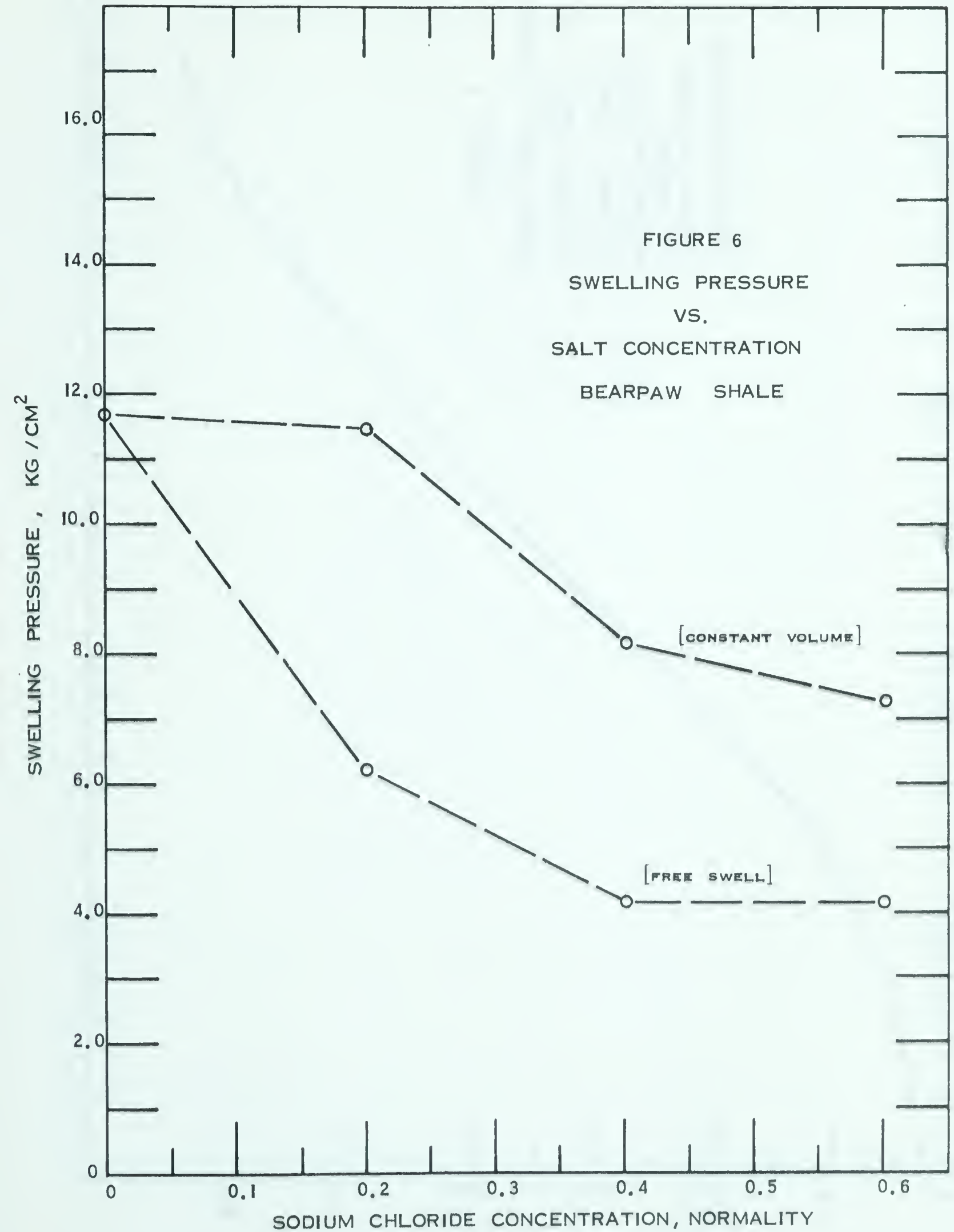
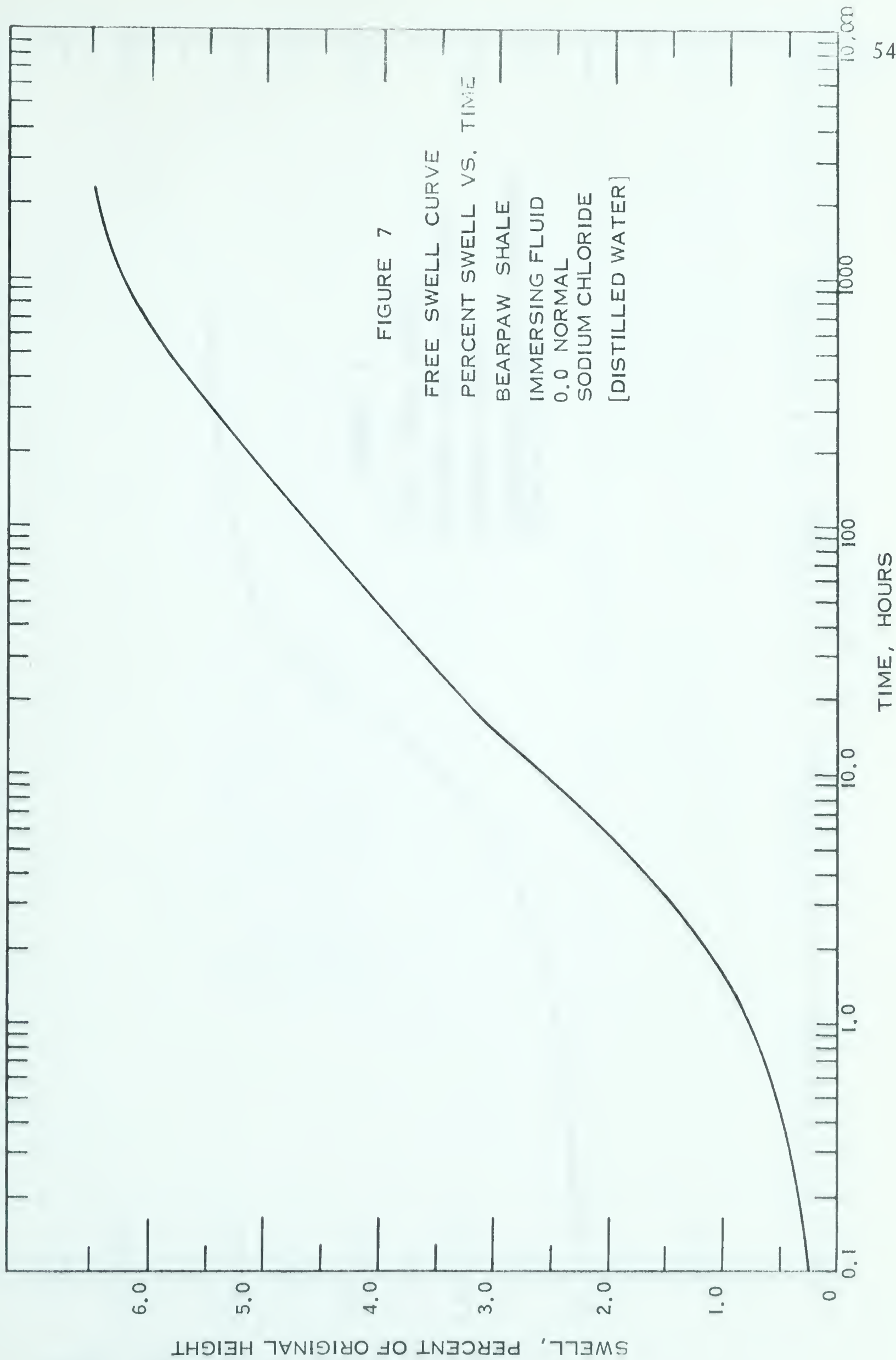
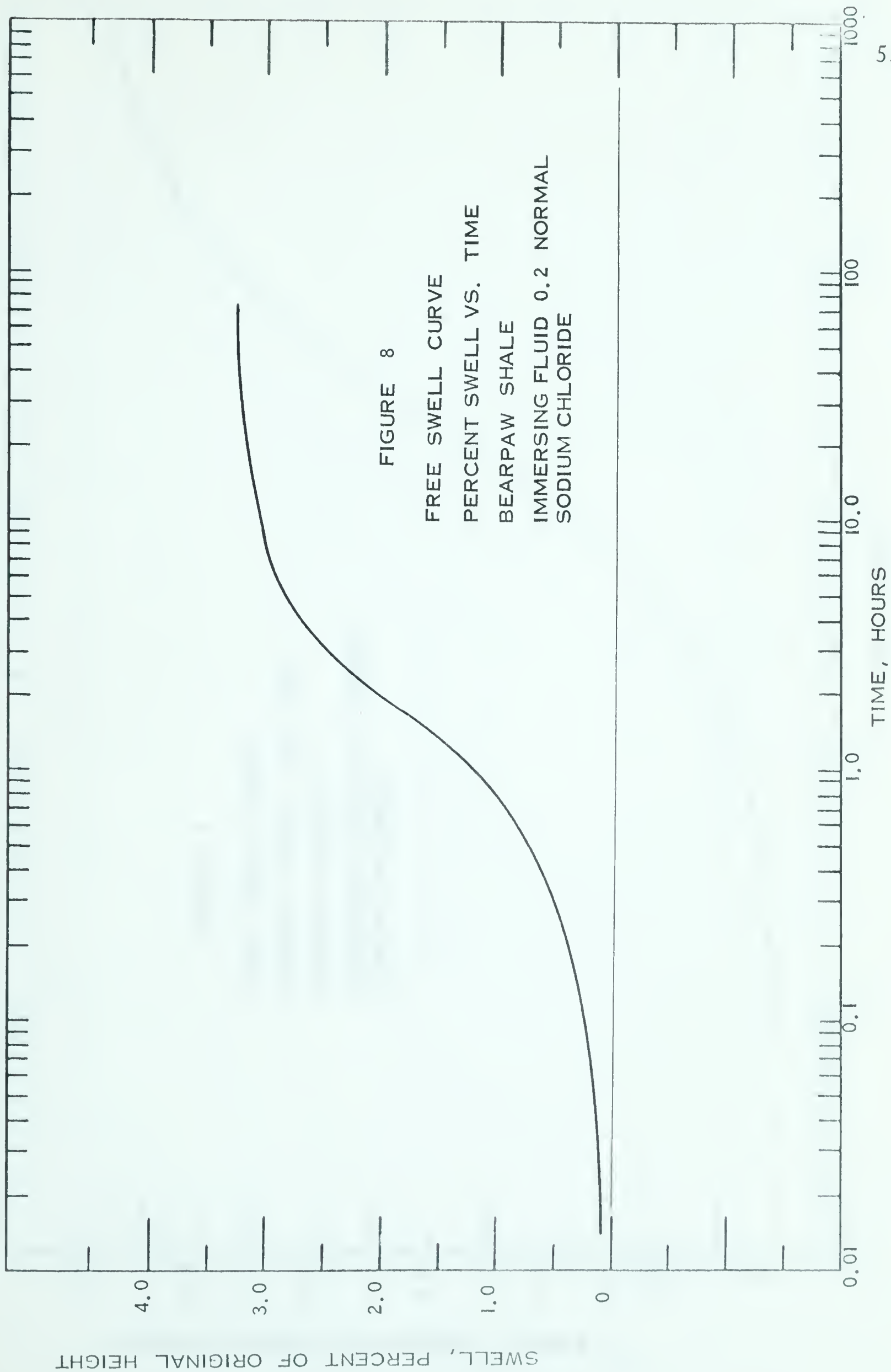
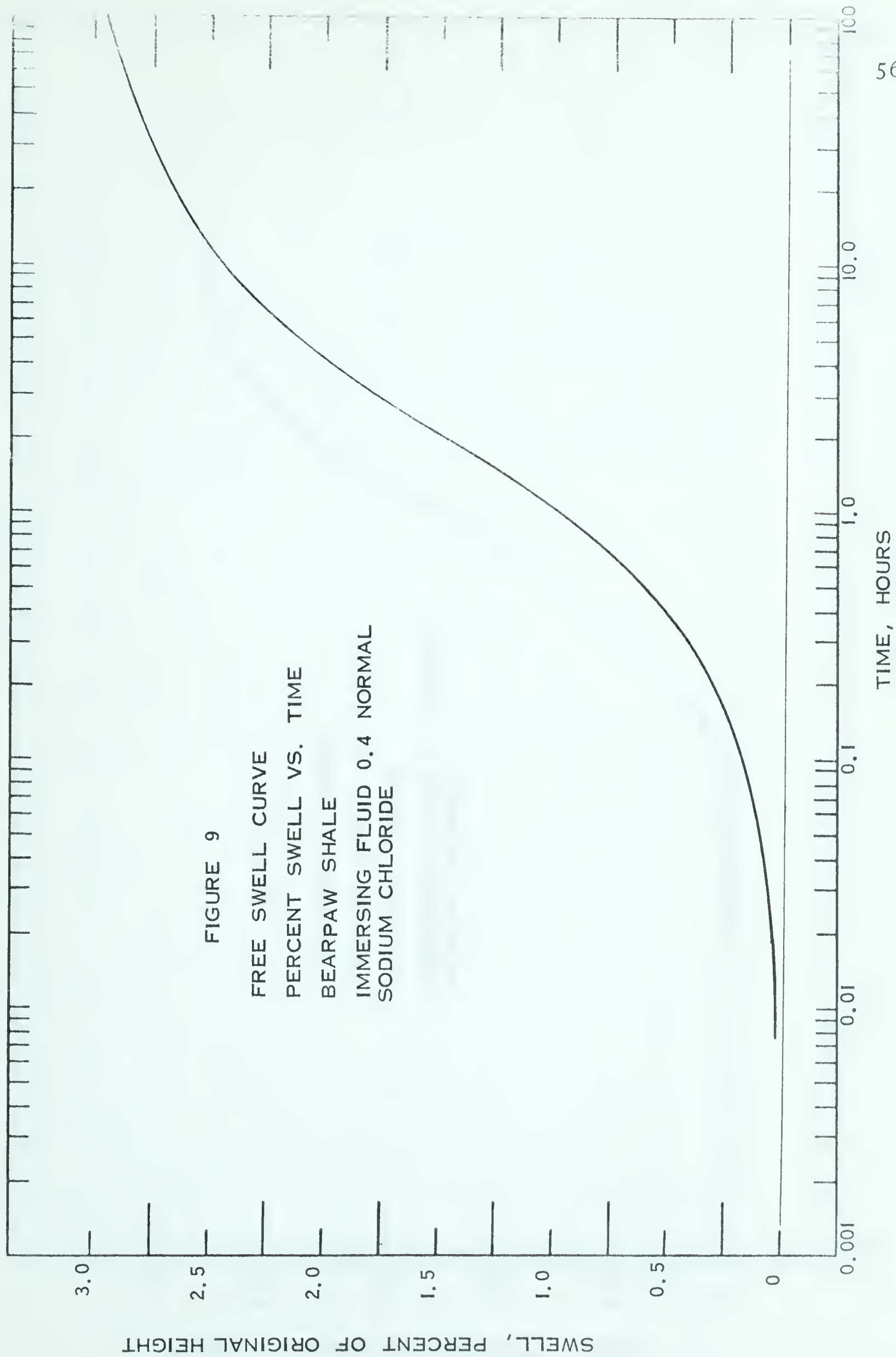


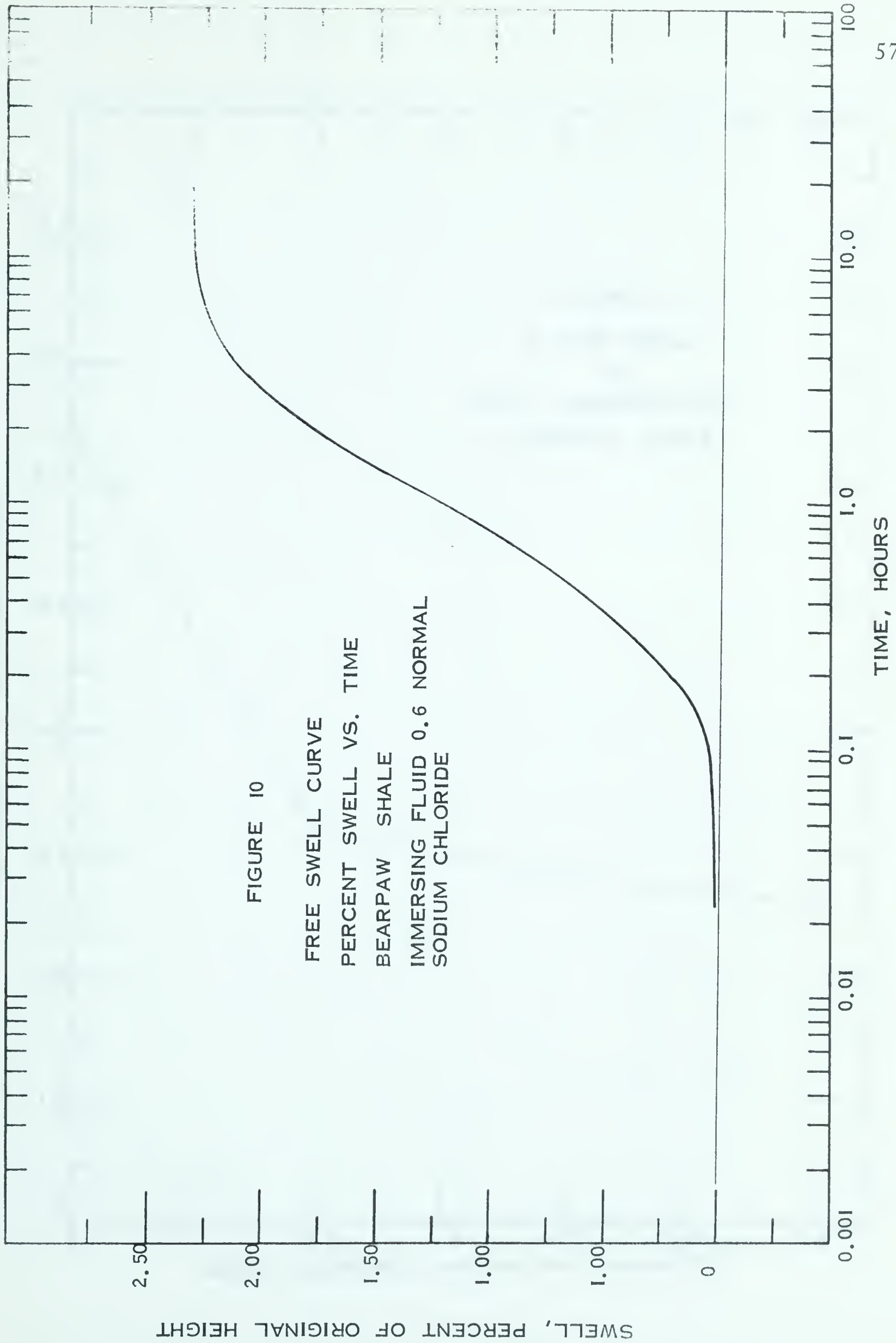
FIGURE 6
SWELLING PRESSURE
VS.
SALT CONCENTRATION
BEARPAW SHALE

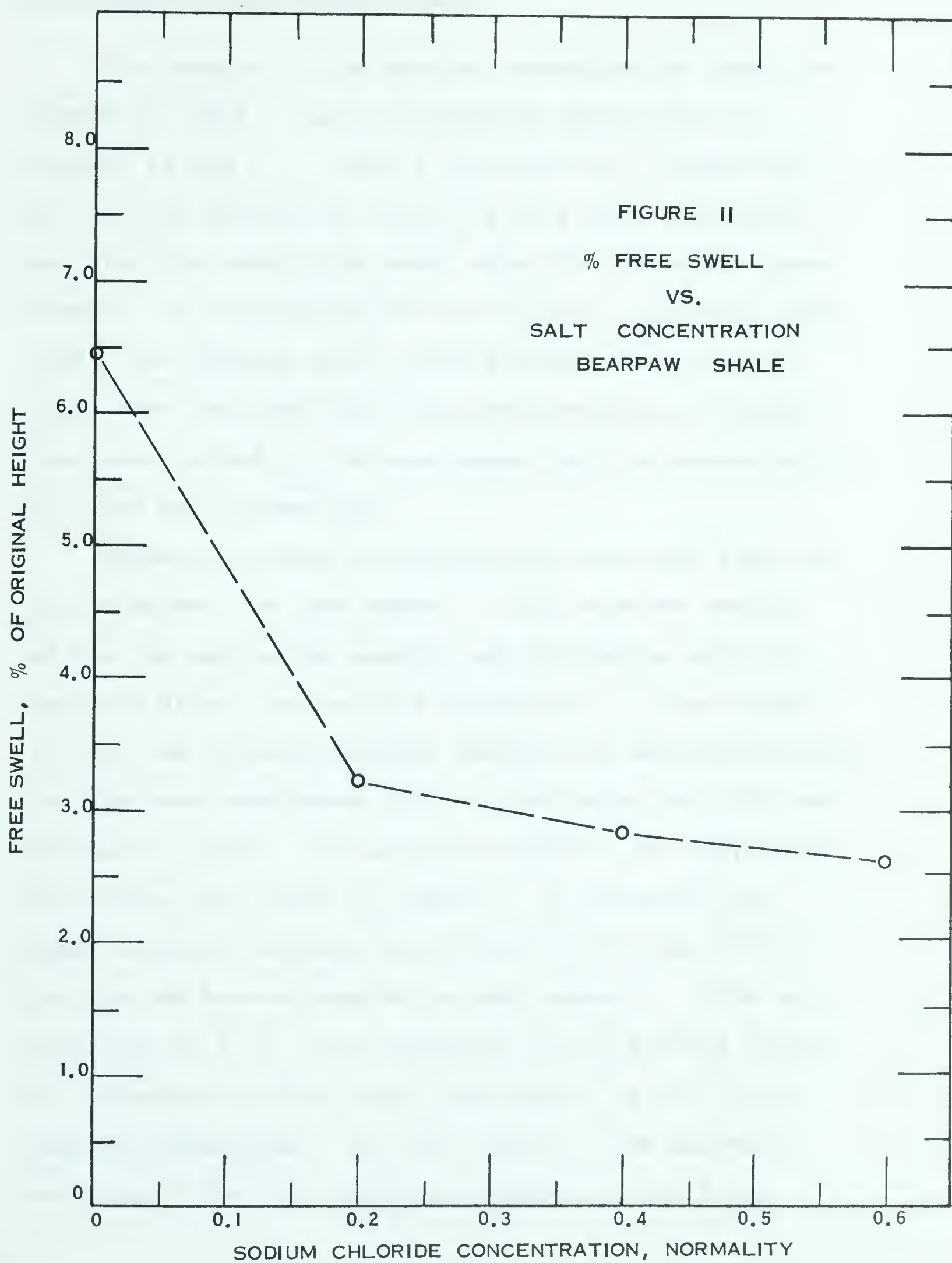












3.5 Special Consolidation Tests

The results of the special consolidation tests are listed in TABLE V and are presented graphically in FIGURES 12 and 13. TABLE V lists the soil properties of the test samples in so far as they were determined and also the conditions under which the tests were performed. As in the case of the standard laboratory swell tests, the samples used in the special consolidation tests were obtained from the same bore hole and depth. They were carved in the same manner and had approximately the same soil properties.

Moisture content determinations were made from the soil samples. In test number 1, the moisture content of the top and bottom samples was determined both before and after the test was completed. In test number 2, only the initial moisture content was determined since the test was terminated without continuing into the consolidation phase. The moisture contents and degrees of saturation are listed in TABLE V. It is noted that these moisture contents vary from 23.99 to 24.38 for the top and bottom samples in test number 1. This is a variation of 1.6%. The variation in the initial degree of saturation between these two samples is 3.0% from complete saturation. In test number 2, the moisture contents of the top and bottom samples varied from

24.16% to 24.51%. This is a variation of 1.5%. The initial degree of saturation did not vary more than 6.2% from complete saturation.

The graphical results of the special consolidation tests are presented by plotting the variables that could be measured against the time-duration of the test. The variables that were measured include total stress, expressed in kg/cm^2 , volume change, expressed as vertical dial deflection in inches, and pore-water pressure, expressed in kg/cm^2 . The values of these parameters were plotted as ordinates against duration of the test as abscissa.

Although the special consolidation tests were to be carried out under constant volume conditions, some volume change occurred, predominantly during the initial stages of the tests. During these initial stages, load increments were added at very short time intervals because the sample had a great affinity for water. As the loads were applied, the dial indicator registered a positive deflection. This indicated that the sample was compressing. However, due to the compressibility of the test apparatus and of the filter paper, the net deflection was negative. Thus, each time a load was applied, the samples swelled a small amount. In both tests, the majority of the swell took place within 15 hours of the test's beginning and did not exceed

0.0055 inches as measured by the vertical dial indicator.

An attempt was made to conduct both special consolidation tests in the same manner. After the completion of test number 1, a second test was performed under the same conditions in order to duplicate results. Only then would the test results have meaning.

FIGURE 12 presents the results of special consolidation test number 1. The initial confining pressure for this test was 0.184 kg/cm^2 . The samples were allowed access to the electrolyte solution and were loaded as the tendency for swell became evident.

It is noted that a total stress of 2.36 kg/cm^2 was added to the sample during the first 20 hours of the test. The addition of this load to the samples was accomplished in increments. Each time the vertical dial indicator registered a deflection of 0.0002 inches, an additional load was applied. After 20 hours, the tendency for volume increase had effectively dissipated. Only 0.06 kg/cm^2 was necessary to prohibit swelling during the next 28 hours.

During the first 48 hours of the test, the samples had access to the 0.3 normal sodium solution. No pore pressures were measured during this time period because the system was opened to the atmosphere. After 48 hours, it was assumed that an equilibrium state had been reached

in which the total stress acting on the samples was counter-balanced by the samples' tendency for swell. This assumption was made on the basis that no further increase in volume was evident from the vertical dial indicator. Theoretically, any additional load would have caused consolidation to occur.

The leaching process took place 48 hours after the beginning of the test. The original electrolyte solution was replaced by distilled water and the center porous plate was flushed. The leaching process took place in approximately 30 minutes. It was noted that a tendency for swell occurred almost immediately. The samples' desire for water was reflected by a change in reading of the vertical dial indicator. Loads were once again applied to inhibit any volume change. In addition, pore pressure transducer readings were recorded since the test system was now pressure tight.

From FIGURE 12, it can be seen that an increase in total stress of 0.85 kg/cm^2 was necessary to keep the samples from swelling. Loading continued for an additional 70 hours. At the end of this time, the tendency for swell had once again become negligible. No effective volume change occurred during the 70-hour loading period until the last load was applied. Pore pressures that were deduced from the strain gauge indicator were only slightly negative.

FIGURE 13 presents the results of special consolidation test number 2. Essentially the same results were obtained from this test as were obtained from test number 1. When the initial electrolyte solution was allowed to the soil, the sample showed an immediate desire to increase in volume. Loads were added to the sample whenever the vertical dial indicator registered a deflection of 0.0001 inches. A volume increase took place during the first 15 hours of the test, but did not exceed 0.0055 inches as measured by the vertical dial indicator. An equilibrium condition was reached where the sample no longer had a tendency for swell. Leaching at this time was performed in the same manner. Upon displacement of the electrolyte solution with distilled water, a tendency for swell was again evident from deflections in the vertical dial indicator. Additional loads were applied until the sample had no further desire for volume increase. From the point in the progress of the test where leaching took place, pore pressures that were measured were slightly negative.

Although the action of tests 1 and 2 was essentially the same, some significant differences were noted. In test number 1, the total load necessary to inhibit swelling under the 0.3 normal sodium solution was 2.32 kg/cm². The tendency for swell had effectively dissipated

after 32 hours. The leaching process was undertaken at the 48 hour time point and the tendency for swell was immediately evident. Total increase in load necessary to inhibit further swelling was 0.85 kg/cm^2 . The tendency for swell under distilled water conditions was negligible after 68 hours from leaching.

In test number 2, the total stress necessary to inhibit swelling under the sodium solution was 2.80 kg/cm^2 . This is an increase of 0.48 kg/cm^2 over that required in test number 1. The tendency for swell had effectively dissipated after 36 hours. Leaching was not conducted until the 6-day period. This was necessary to be quite sure that the samples had no further tendency for swell. From the 36 hour time point until the 6 day time point when leaching occurred, a slight tendency for swell developed. This tendency was inhibited by the addition of two small loads (0.03 kg/cm^2). After leaching with distilled water, the tendency for swell was not so strong as in test number 1. Loads added to the samples to inhibit swelling were more evenly spaced in time. The increase in total load was 1.07 kg/cm^2 . This is an increase of 0.22 kg/cm^2 from test number 1, and is probably insignificant. The tendency for swell under distilled water conditions did not become negligible until 6 days (144 hours) after leaching took place.

This is an increase of 46 hours over that required in test number 1.

The differences in the two special consolidation tests may or may not be significant to a plausible interpretation of the test results. However, it is the opinion of the author that the results obtained from test number 2 are more indicative of the behavior of the soil under test conditions. This opinion is attributed to the fact that test procedures were more refined in test number 2.

TABLE V

SUMMARY OF RESULTS, SPECIAL CONSOLIDATION TESTS

TEST NUMBER	INITIAL CONFINING PRESSURE KG/CM ²	** SALT CONCENTRATION OF IMMERSING FLUID NORMALITY BEFORE AFTER LEACHING	MAXIMUM TOTAL STRESS ON SAMPLES KG/CM ² BEFORE AFTER LEACHING	MAXIMUM PORE WATER PRESSURE KG/CM ² BEFORE AFTER LEACHING	MAXIMUM DIAL DEFLECTION INCHES BEFORE AFTER LEACHING	* MOISTURE CONTENT INITIAL FINAL %	* INITIAL DEGREE OF SATURATION %
1	0.184	0.300 0	2.32 3.17	0 -0.05	0.0052 0.0055	24.18 28.37	98.26
2	0.184	0.300 0	2.80 3.87	0 -0.10	0.0056 0.0056	24.33 -	95.29

* AVERAGE OF TOP AND BOTTOM SAMPLES

** REFERS TO SODIUM CONTENT, OTHER SOLUBLE SALTS WERE AS IN ORIGINAL PORE WATER ANALYSIS

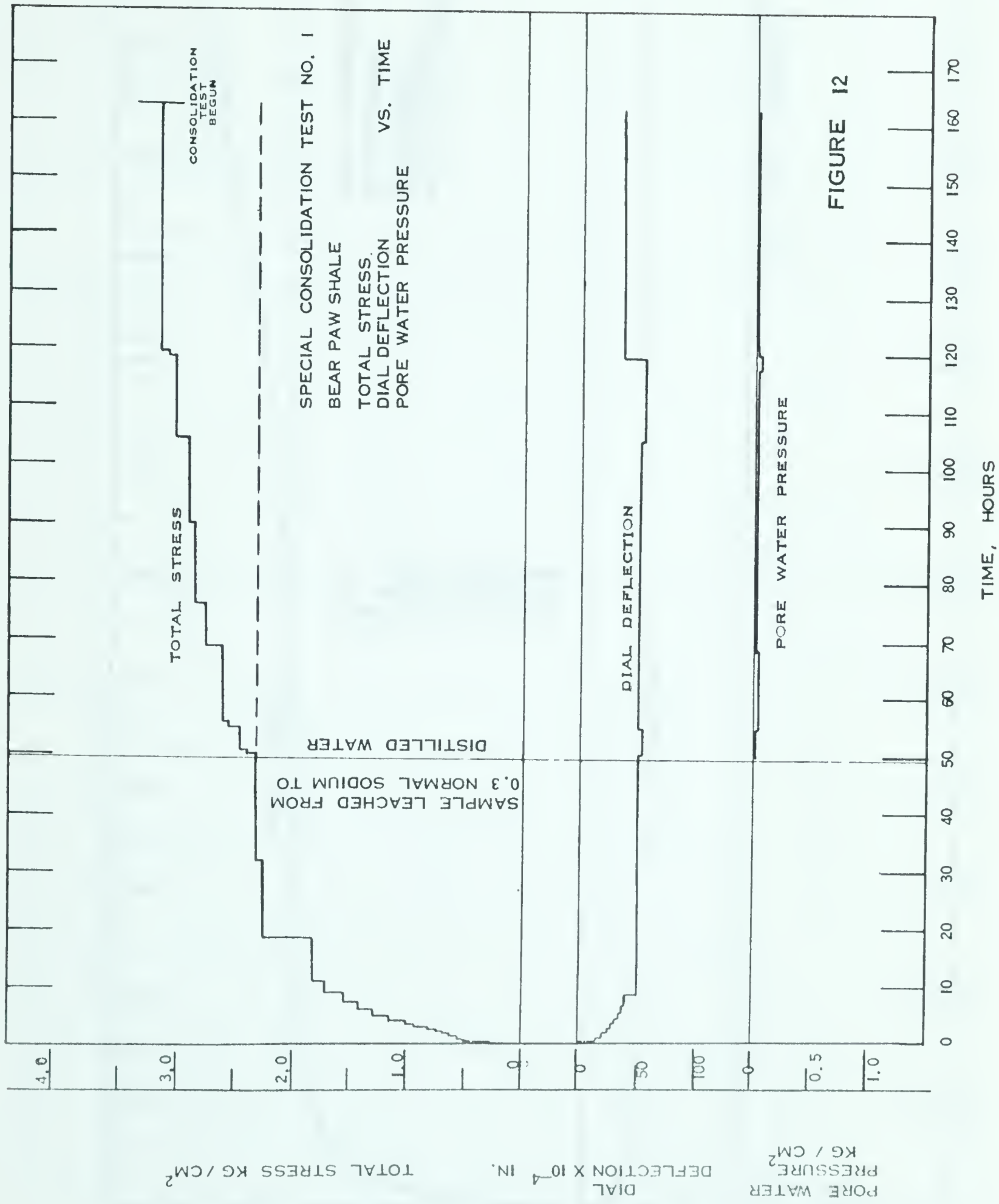
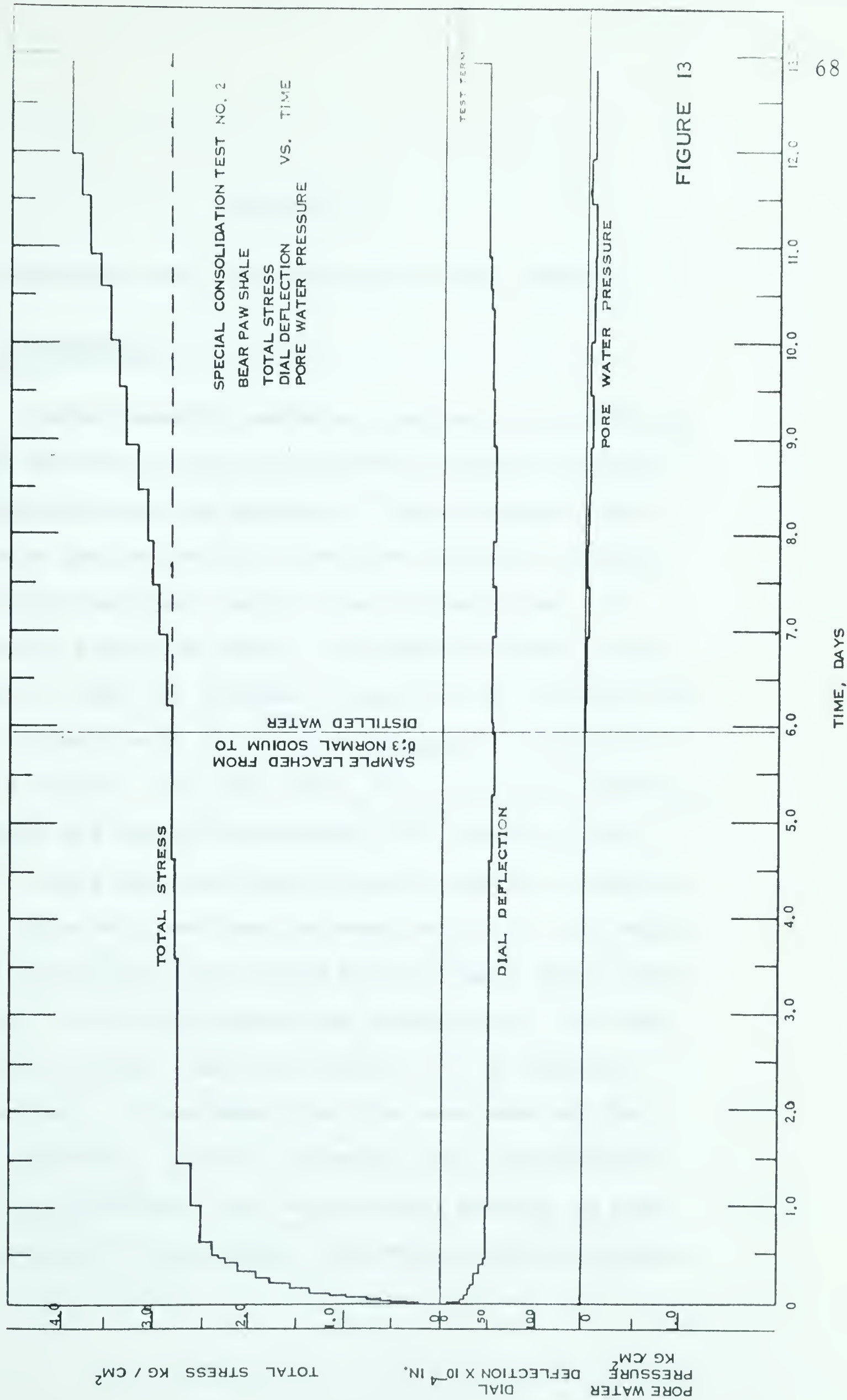


FIGURE 12



CHAPTER IV

DISCUSSION AND INTERPRETATION OF TEST RESULTS

4.1 Introduction

In many research projects concerned with swelling soils, the use of clay suspensions, remolded samples, or homionic soils is employed. The principal value of their use is that the structure, moisture content, and cation exchange complex can be controlled. In naturally occurring soils, such factors cannot be controlled. They are inherent properties of the soil and must be dealt with in terms of changes in place of absolute values. At best, these factors can be considered constant and hence controllable to a certain extent.

In this research program, soil samples of Bearpaw Shale were obtained from the same bore hole, and depth. All test samples were carved from the same small block of soil. All test samples had approximately the same moisture content, and were within $\pm 7\%$ of complete saturation. It was hoped that the remainder of the soil properties, such as structure and orientation of the clay particles, were sufficiently similar so that any changes in the swelling and stress characteristics

with changes in the physico-chemical environment could be evaluated. The discussion and interpretation of the test results are based on this assumption.

4.2 Results of Standard Laboratory Swell Tests

The factors which affect swelling in fine-grained soils have been outlined by Lambe (1959), by Ladd (1960), and by Seed, et al, (1962). These factors are presented here for reference in discussion. They are:

1. The stress history of the soil.
2. The initial moisture content of the soil.
3. The structure of the soil.
4. The composition of the clay minerals.
5. Environmental factors, such as temperature, pressure and time.
6. The nature of the pore fluid.

The first three of the above factors are considered by Seed et al, (1962) to be mechanical components. The remaining three are classified as physico-chemical components. It is generally agreed that expansion of a fine-grained soil is attributable to the combined effect of all these factors.

The swelling investigation in this project was concerned with the sixth factor listed above. The

qualitative effect of the remaining factors on the test results may be accounted for in the following manner.

The stress history of Bearpaw Shale has been outlined by Peterson (1954). It is estimated that at least 2000 feet of sediment has been eroded from the site at which the test samples were obtained. The soil has therefore been subjected to a large reduction in confining pressure and has a tendency for swell. Peterson & Peters (1963) consider that elastic rebound due to removal of overburden takes place immediately. They conclude that swelling due to geological unloading is complete or so slow that it cannot be detected.

The effect of the initial moisture content is considered to be constant for all test samples. All initial moisture contents were approximately the same, and all were above the shrinkage limit. In general, the effect of initial moisture content on swelling properties is most prevalent when below the liquid limit. Most authorities, Lambe (1960), Ladd (1960), believe that the initial moisture content has the greatest influence on swelling when between the liquid limit and the shrinkage limit.

The effect of structure on the swelling properties of Bearpaw Shale cannot be evaluated since it was not

controlled. At best, the effect can be considered to be constant for all test samples.

The composition of the clay minerals certainly contributes to the amount of swell and to the swelling pressures developed in Bearpaw Shale. Mineralogic tests indicated that the predominant clay mineral constituent was montmorillonite. Tests to determine the cation exchange characteristics indicated that the predominant exchangeable cation was sodium. It is known that this particular combination of clay mineral and exchangeable cation has the greatest potential for swell of all combinations of exchangeable cations and clay minerals (Lambe, 1959).

Environmental factors such as pressure, temperature, and time can be significant when carried to extremes. The effect of confining pressure on swelling characteristics has been investigated by Seed, et al, (1962). It was shown that the amount of swell that takes place is greatly dependent on the initial confining pressure. In this research project, the effect of initial confining pressure can be considered to be constant since samples tested in free swell were confined under approximately the same pressure ($0.184 - 0.195 \text{ kg/cm}^2$). In the constant-volume swell tests, the initial confining pressure has no effect since no volume change is allowed.

The effect of temperature on swelling properties is considered negligible for this project. The relationship between temperature and swelling pressure is not completely understood. Warkentin, et al, (1962) report that swelling pressures decrease with decrease in temperature.

The remaining environmental factor (time) is of considerable importance in this or any program investigating swelling phenomena. Time effects cannot be neglected in swelling since the amount of swell and subsequent swelling pressure are directly dependent on how long a soil takes to satisfy its desire for water.

Seed, et al, (1962) have shown that the swelling process is analogous to the process of consolidation in the effect that the imbibition of water is controlled by the permeability of the soil. The shape of a time-swell curve is essentially the reverse of a time-consolidation curve. This can be seen in FIGURES 7, 8, 9, and 10.

The last factor considered to affect swelling in fine-grained soils was investigated in this research project. The effect of a variation in the nature of the pore fluid was conducted to evaluate the osmotic pressure theory as it applies to swelling in Bearpaw Shale.

In light of the factors affecting swelling, it

appears reasonable to believe that most of the changes recorded in the amount of swell and in swelling pressure are due to changes of an osmotic pressure since only the electrolyte concentration of the immersing fluid was varied. Since it is necessary to deal with changes in place of absolute values, a reference datum must be chosen. The reference datum is the amount of swell and the swelling pressure of the test sample immersed in distilled water.

4.3 Osmotic Theory and Free-Swell Test Results

In TABLE IV is presented a summary of the free-swell test results. The pertinent results are repeated below in TABLE VI for convenience in discussion.

TABLE VI
FREE-SWELL TEST RESULTS

Salt* Concentration of Immersing Fluid Normality	Swelling Pressure kg/cm ²	Swell Percent of Original Height	Salt** Concentration of Pore Water Normality
0.0	11.7	6.44	0.143
0.2	6.2	3.26	0.143
0.4	4.2	2.93	0.143
0.6	4.3	2.33	0.143

* NaCl

** Includes Only The Concentration of Sodium Salts

Since the sample immersed in distilled water was chosen as the reference datum, the swelling pressure and the percent of swell represent the combined effect of all factors that influence swelling properties. The reduction in swelling pressures and in percent swell may be explained by a consideration of the adsorbed water layers.

In clay soils containing a predominant amount of sodium montmorillonite as the clay mineral, it is believed that over 90% of the water present in the soil may be adsorbed water (Grimm, 1962). When a clay soil is immersed in distilled water, the difference in salt concentration of the double layer water and that of the immersing fluid is a maximum. Under these conditions, the greatest osmotic pressure should be developed. As the electrolyte concentration of the immersing fluid is increased, the osmotic pressure is reduced since the difference in electrolyte concentration is decreased. When a concentration difference no longer exists, the osmotic pressure between the adsorbed water and the immersing fluid should be zero.

For all samples tested, the sodium concentration of the pore water was 0.143 normal. Thus, it appears reasonable to conclude that the difference in swelling pressures between the sample immersed in distilled water and that immersed in 0.2 normal sodium chloride is associated with the osmotic pressure.

The difference in swelling pressure between the sample immersed in 0.2 normal sodium chloride and that immersed in 0.4 normal does not seem rationally explained by osmosis. It may however, be attributable to some additional factor. Possible explanations for this difference are:

1. A difference in the soil structure between samples.
2. The influence of time effects on the swelling pressure.

It is noted that the swelling pressures for the test samples immersed in 0.4 and 0.6 normal sodium chloride are essentially the same. Since the electrolyte concentrations of the immersing fluids are much greater than that existing in the pore water, it is reasonable to expect that they should be similar. These swelling pressures (4.2 and 4.3 kg/cm²) are therefore assumed to reflect all factors contributing to swelling exclusive of osmotic effects. As a consequence, the difference between these latter swelling pressures and those recorded for the sample immersed in distilled water might be attributed to osmotic swelling.

A similar argument can be offered to explain the differences in the percent swell. It is known that the addition of an electrolyte depresses the double

layer (Scott, 1963). In free-swell tests in which the concentrations of the immersing fluids are varied, the percent swell represents an equilibrium between the depression of the double layer and the desire for imbibition of water by the soil (Lambe, 1959).

Percent swell is further complicated by the influence of time. Seed, et al, (1962) states:

"The long time required for the development of full swell or swell pressure is a result of the time required to effect the stress transfer in the water from a large negative value existing before exposure of the sample to the water up to the zero value that must exist throughout the system at equilibrium."

The impervious nature of Bearpaw Shale probably precludes attaining equilibrium in a reasonable duration of time. This is evident from FIGURE 7 which shows a free swell-time curve extending over a period exceeding 2000 hours. The desire for water at this time had not yet been satisfied.

4.4. Osmotic Theory and Constant-Volume Test Results

In the constant-volume swell test, the action of osmosis between adsorbed water and immersing fluid is more easily definable. In this test, the osmotic pressure that is developed from differences in electrolyte

concentrations is theoretically not dissipated during the test since no volume change is allowed. Osmotic pressures should develop at the outset of the test and should remain constant so long as no volume change is allowed (Hardy, 1965). If a variation in the immersing fluid is allowed, a variation in the osmotic pressure should be reflected by variations in the pressure at which consolidation occurs.

The pertinent results of the constant-volume swell tests are presented below in TABLE VII for purposes of discussion.

TABLE VII
CONSTANT-VOLUME SWELL TEST RESULTS

Salt* Concentration of Immersing Fluid Normality	Swelling Pressure kg/cm ²	Salt** Concentration of original Pore Water Normality
0.0	11.7	0.143
0.2	11.5	0.143
0.4	8.0	0.143
0.6	7.4	0.143

* NaCl
** Includes Only The Concentration of Sodium Salts

In general, the same trend was recorded in the constant-volume tests as in the free-swell tests. A decrease of swelling pressure occurred with an increase in electrolyte concentration of the immersing fluid (FIGURE 6).

It was noted that a decrease in swelling pressure was not observed between the test samples immersed in distilled water and in 0.2 normal sodium chloride. This appears to be an anomaly with respect to the osmotic theory. For the samples immersed in 0.4 and 0.6 normal sodium chloride, a decrease in swelling pressure was observed from the sample immersed in distilled water. The differences in these swelling pressures most likely reflect the osmotic effect. The swelling pressure for the sample immersed in distilled water probably reflects all factors affecting swelling and is in agreement with the free-swell tests.

It is not known whether experimental errors may account for the anomaly mentioned above, but the manner in which swelling pressures are defined in constant-volume swell tests may contribute to an explanation.

In the constant-volume tests performed by the author, complete control of volume changes was not possible due to the compressibility of the testing apparatus. It was therefore necessary to allow a small

change (swell) to take place and to define the swelling pressure as that pressure at which the consolidation curve crosses the zero-dial deflection line. Because of this, the swelling pressure is very susceptible to small changes in swell since each increment increases the ordinate of the dial-deflection-curve. Where the curve crosses the zero-dial-deflection line is somewhat arbitrary. A graphical interpretation does not allow for extreme accuracy.

A comparison of swelling pressures obtained from the free-swell tests and from the constant-volume swell tests shows that the swelling pressures from the latter tests are larger than those from the former. (FIGURE 6). This appears to be an anomaly with respect to much of the research done on swelling pressures. In general, constant-volume swell tests usually give values for swelling pressures that are lower than those from free-swell tests. However, Hardy & Dyregrov (1962) reported that for some overconsolidated clays, the constant-volume swelling pressure exceeded the free-swell swelling pressure.

The anomaly observed in the author's test results is believed due to the compressibility of the testing apparatus. For all swell tests performed, corrections for the compressibility of both the apparatus and the

filter paper were applied. The application of these corrections resulted in constant-volume swelling pressures that were higher than the free-swell swelling pressures. If the corrections are not applied, the reverse is true. This can be seen from FIGURE 14.

The results of the constant-volume swell tests determined in this study have convinced the author that a new approach to the measurement of constant-volume swelling pressures is desirable. Much research has been conducted on constant-volume swelling. For example; Ladd (1960), Seed et al, (1962), and Hardy & Dyregrov (1962). In most of this work, no information concerning small volume changes during constant-volume tests has been presented. However, it is known that a small increase in volume considerably reduces swelling pressures (Seed et al, 1962). It is questionable that pressures defined under no-volume-change conditions actually have been measured. Certainly they are not obtainable in conventional laboratory equipment.

FIGURE 14

COMPARISON OF CONSTANT-VOLUME
CONSOLIDATION CURVES WITH
AND WITHOUT COMPRESSIBILITY
CORRECTION APPLIED

BEARPAW SHALE

IMMERSING FLUID 0.0 NORMAL
SODIUM CHLORIDE
[DISTILLED WATER]

SWELLING PRESSURE
11.7 KG/CM²
WITH COMPRESSIBILITY
CORRECTION APPLIED

FREE SWELL

SWELLING PRESSURE 2.05 KG/CM²
WITHOUT COMPRESSIBILITY
CORRECTION APPLIED

DIAL DEFLECTION $\times 10^{-4}$ INCHES

PRESSURE KG/CM²

4.5 Results of Special Consolidation Tests

It was pointed out in Chapter I, that the initial aim of this project was to investigate what effect a variation in the physico-chemical environment has on effective stresses during a one-dimensional consolidation test. To accomplish this aim, a highly swelling, undisturbed soil was subjected to a constant-volume swell test in which the chemistry of the immersing fluid was controlled. An attempt was made to control the amount of volume change taking place and to measure pore-water pressures during critical periods in the progress of the test.

The results of the two tests were summarized in TABLE V and presented graphically in FIGURES 12 and 13. These results will be discussed in the following sections.

4.6 Effective Stresses During Constant Volume Tests

It is first appropriate to examine the stress conditions of a saturated soil during a constant-volume swell test. When a saturated soil sample is confined in a consolidometer, the stresses acting on it can be defined in terms of the effective stress concept. Initially there is a very small total stress acting on the sample. This is the confining

pressure. The soil water is in tension at this time due to the release of overburden pressure from the in-place conditions in the field. Since a small total stress is acting on the sample, the effective stress must be approximately equal to the negative pore-water pressure in accordance with equation 4.1.

$$\sigma' = \sigma - U \quad 4.1$$

$$\text{For } \sigma \cong 0: \sigma' \cong -U$$

$$\text{For } U \cong 0: \sigma' \cong \sigma$$

where σ = the total stress acting on the sample

σ' = the effective stress acting on the soil sample

U = the pore-water pressure acting in the water phase of the soil water system.

When the sample is allowed contact with an immersing fluid, the surface capillary forces are destroyed and the soil imbibes water (Mitchell, 1960). To prevent this imbibition of water an increase in total stress is necessary. If no volume change is allowed, the increase in total stress must be accompanied by a corresponding decrease in the negative pore-water pressure. The effective stress has remained constant and will continue to do so as long as no volume change is allowed. The tendency to imbibe water or swell cannot be maintained indefinitely. At some value of total stress, the negative pore-water pressure will be zero

(Seed et al, 1962). At this point in the progress of the test, the total stress acting on the sample is the effective stress in accordance with equation 4.1. If an increase in total stress occurs, a positive pore-water pressure will result. Consolidation will take place with a subsequent increase in effective stress.

The character of the stress conditions outlined above should apply to any swelling soil in a constant-volume swell test. Usually the immersing fluid is distilled water and the test is carried through the no-volume-change period and into the consolidation phase.

In the special consolidation tests performed in this project, the concentration of the immersing fluid was changed before consolidation took place. It was thought that the point was determined at which the total stress acting on the sample was equal to the effective stress. At this point, the pore-water pressure is theoretically zero. Had pore pressure measurements been possible from the onset of the test, this matter would not be open to question. However, pore pressures were not obtained until just prior to the leaching process because the test system was opened to the atmosphere. Pore pressure measurements at that time indicated that the pressure was nearly

zero. The soil exhibited no further desire for swell.

After a change in the immersing fluid had occurred (leaching with distilled water) an additional demand for water was observed. Once again, an increase in total stress was necessary to prevent the imbibition of water into the soil. In both tests, this additional total stress was approximately 1.0 kg/cm^2 . No volume change was observed during this loading period, and pore pressures that were measured were slightly negative.

Two possible explanations of the behavior of the soil after leaching had taken place seem to be available. These are outlined in the following paragraphs.

4.7 Limitations of The Testing Procedure

One possible explanation of the results obtained from the special consolidation tests may be found in the limitations of the testing procedure. An addition of total stress was necessary to prohibit swelling after leaching had taken place. This increase in total stress could possibly be necessary because the desire of the soil for swelling, developed under conditions of the original immersing fluid, was not yet satisfied. This should manifest itself in a negative pore-water pressure before leaching occurred. However, the pore pressures that were measured immediately before and after leaching took place were nearly zero.

There exists some question as to whether pore pressures can be measured at all in material such as the Bearpaw Shale. Certainly the low permeability of the soil and the fact that over 90% of the pore water may be adsorbed influence any attempt to measure pore pressure. Dahlman (1965) has shown that for homionic sodium modifications, pore pressure reactions may be as small as 14%. Hardy (1965) has maintained that pore pressure measurements cannot be relied upon in heavily overconsolidated soils. This question is still open to conjecture.

The fact that pore pressure measurements may be questionable does not, however, rule out the possibility of obtaining that point in a constant-volume swell test at which the total stress is equal to the effective stress. A second, and perhaps less reliable method is available from a consideration of the tendency for swell. In test number 2, the process of leaching was purposely delayed for a period of 6 days to allow the tendency for swell to become negligible. During this period very little volume increase was observed. The addition of two small increments of total stress (0.03 kg/cm^2) inhibited any further tendency of the soil to swell. This can be seen in FIGURE 13. At the 6-day period, it was assumed that the total stress

was equal to the effective stress and leaching was performed. Based on the observation of the behavior of the soil before and after leaching occurred, it appeared that this condition was obtained. Therefore, the author rules out the possibility that the test procedures explain the behavior of the soil.

4.8 A Consideration of the Osmotic Pressure Theory

A second possible explanation for the behavior of the soil after leaching is concerned with the osmotic pressure theory. At the outset of the special consolidation tests, the immersing fluid had a greater concentration of sodium salts than did the pore water of the soil. The remaining soluble salts in both areas were identical. Therefore, during the period before leaching took place, no osmotic pressure would be expected to develop. The tendency of the soil for swell during this period would be due only to those factors contributing to swell exclusive of osmotic effects. The major portion of this tendency for swell was probably due to release of capillary tension. The behavior of the soil up to the point of leaching can therefore be explained on the basis of effective stresses in the constant-volume swell test in accordance with equation 4.1.

Upon leaching with distilled water, it is reasonable to suppose that an osmotic pressure was created between

the distilled water and the pore water of the soil. The development of an osmotic pressure would tend to cause swell since it acts to increase particle spacing (Lambe, 1959). An increase in total stress would be necessary to prohibit the desire of soil for water. As was explained previously, the increase in total stress on the soil could not continue indefinitely, but the amount of increase should reflect the osmotic pressure tending to cause swell. As the total stress increase approached the osmotic pressure in magnitude, the tendency for swell would become less, eventually reaching zero. When the tendency for swell no longer existed, the increase in total stress on the sample over that existing just prior to leaching should be equivalent to the osmotic swelling pressure.

A consideration of the above hypothesis presupposes that no pore-water pressure was created by an increase in total stress over that existing prior to leaching. The increase in total stress is counter-balanced by the osmotic pressure tending to cause swell. In light of the test results, this hypothesis is a very plausible explanation.

It can be argued that pore-water pressures are created in the soil by the additional total stress, but that they are not measurable. To investigate this

argument, the author conducted a pore pressure reaction test on sample number 2 after the constant-volume test was completed. The pertinent data from this reaction test are given in TABLE VIII.

It is noted that a "B" parameter value of approximately 0.6 was recorded for all pressures involved. After a time lapse of 60 minutes, the highest reaction obtained was 0.602. It is important to note that this test was conducted at a time when sample number 2 had no further desire for swell. This time was approximately 7 days after leaching had occurred. According to the hypothesis under discussion, the total stress existing on the sample prior to the pore pressure reaction test was counterbalanced by the osmotic pressure. The addition of total stress increments during the pore pressure reaction test should have resulted in positive pore-water pressures. According to TABLE VIII the fact that pore pressures existed during the reaction test cannot be disputed. The pore pressure reactions are low, but they are consistent with results obtained by Dahlman (1965) on consolidated homionic sodium modifications, and with Hollingshead (1965) for the identical Bearpaw Shale employed for the present tests.

TABLE VIII

PORE PRESSURE REACTION TEST RESULTS

Applied Pressure psi	Time Allowed For Reaction To Take Place Minutes	Pore Pressure Recorded psi	B
13.49	15	8.35	0.619
26.85	15	15.90	0.592
	30	16.10	0.599
26.85	15	15.66	0.583
	30	16.00	0.596
26.85	15	15.95	0.593
	30	16.11	0.599
	60	16.19	0.602

It is not the author's intent to discuss the reasons why the B parameters are low. It seems sufficient to show that pore pressures were present and measurable after the tendency of the soil for swell was dissipated. This point is of considerable importance to the interpretation of the test results. Under the test conditions when the soil exhibited a desire for swell, an applied stress induced no pore water pressures, at least none that were measureable to any extent. Yet, under the same test conditions when the soil no longer had a desire for swell, an applied stress induced pore-water pressures to the

extent that they were measureable. As a result, it is rational to assume that the test measurements are reliable.

From the test results, it therefore appears that during leaching of the soil, an osmotic pressure was created due to the difference in sodium concentration of the soil's pore water and that of the leaching fluid. The development of this osmotic pressure resulted in a tendency of the soil to swell and this tendency was inhibited by an increase in total stress. During the loading period, no pore pressures were measured because the total stress increase was taken by the osmotic pressure. When the total stress increased beyond the magnitude of the osmotic pressure, pore pressures developed and were measurable to a reasonable extent.

At the time of leaching of the soil, the total stress acting on the soil is given by equation 4.1. In test number 1, the total stress acting on the sample just prior to leaching was 2.32 kg/cm^2 , while in test number 2, the total stress acting on the sample prior to leaching was 2.80 kg/cm^2 . After leaching, and subsequent development of osmotic pressures, the total stress acting on samples number 1 and 2 were 3.17 and 3.87 kg/cm^2 respectively. During

the leaching period, and until the end of the tests when the soil no longer exhibited a tendency for swell, the pore-water pressure was slightly negative in both tests. Accordingly, the osmotic pressures developed were 0.85 and 1.07 kg/cm². These values are in good agreement with one another and are probably indicative of the true osmotic pressure developed under the test conditions.

An increase in total stress subsequent to leaching, resulted in no volume change and no measureable positive pore-water pressures. The conclusion seems to be that the total stress increase must be equal to the developed osmotic pressure.

It was mentioned previously (page 84) that the effective stress existing in a soil sample during a constant-volume swell test will remain constant as long as no volume change is allowed. If this assumption is not valid, then the increase in total stress subsequent to the leaching process may be explained by an increase in effective stress. If the effective stress increases with no volume change, no pore-water pressures of any magnitude need be measured. However, the author was unable to locate any information that relates an increase in effective stress to a reduction in pore-water salt concentration. This seems contrary to most of the literature available

on the subject (Thomson, 1963; Rosenqvist, 1955; Bjerrum, et al, 1964).

Olson (1964) considers that leaching of a soil results in a new soil-water system. Effective stresses that exist in a soil before leaching are considered constant as long as leaching does not occur. If a change in the salt concentration of the pore water is allowed, the soil-water system is changed and a new set of stress conditions is present.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In light of the literature available on this subject, and in view of the results obtained from this research program, it appears that the following conclusions are justified for the soil involved.

5.2 Conclusions from the Literature

Swelling properties associated with overconsolidated clay soils are highly dependent on the soils' physico-chemical nature as well as on their mechanical nature. The many factors contributing to these properties are not completely understood at the present time. The mechanisms by which swell takes place are also not completely defined.

It is generally agreed that shearing resistance in fine-grained soils is highly dependent on the physico-chemical environment. Most authorities will readily admit that a reduction in shear strength will be caused by a reduction in the electrolyte concentration of the pore water.

5.3 Conclusions from the Free-Swell Tests

A variation in the electrolyte concentration in which the soil samples were allowed to swell has a considerable effect on the free-swell swelling pressure. This is indicated from FIGURES 2 through 6. The free-swell swelling pressure varied by more than 50% with changes in the electrolyte concentration of the immersing fluid. This variation was not linear. In general, there is a rapid decrease in swelling pressure with an increase in concentration of the immersing fluid from zero to 0.2 normal sodium chloride. The variation in swelling pressure from 0.2 normal sodium chloride to the maximum value studied was smaller and approached a constant value at 0.6 normal.

The amount of swell that took place in the free-swell tests is also dependent on the electrolyte concentration of the immersing fluid. This is evident from FIGURE 11. The variation is not linear, but decreases rapidly at relatively small concentrations of the immersing fluid. At larger concentrations, the percent swell became almost constant.

Both the free-swell pressure and the amount of free-swell are time-dependent. It is concluded that an overconsolidated clay soil must reach an equilibrium stage between desire to swell and time allowed to swell if the swelling pressures are to be evaluated

correctly.

It is also concluded that a measure of the osmotic swelling pressure might be deduced from the free swell tests by conducting these tests at various electrolyte concentrations of the immersing fluid.

5.4 Conclusions from the Constant-Volume Tests

A variation in the electrolyte concentration of the immersing fluid also has a considerable effect on the constant-volume swelling pressure. It appears that a similar relationship exists between constant-volume swelling pressure and concentration of immersing fluid. However, this relationship is not so well defined as in the free-swell tests. Constant-volume swelling pressures are highly dependent on the amount of volume change allowed to occur. It is concluded that constant-volume swell tests are more inaccurate than free-swell tests because of the difficulty in maintaining a constant volume throughout the initial stages of the test.

5.5 Conclusions from the Special Consolidation Tests

It is concluded that a change in the physico-chemical environment does affect the stress conditions existing in a highly overconsolidated clay soil. For the soil involved, it seems that an osmotic pressure

was developed when a change in the electrolyte concentration of the leaching fluid was initiated. Under the environmental conditions studied, an increase in total stress appeared to be counter-balanced by the development of this osmotic pressure. It is concluded that during the leaching period, and during the time period when the osmotic pressure was acting, no pore-water pressure of any consequence was measured.

5.6 Recommendations For Future Research

From a consideration of the experimental research performed by the author, the following recommendations are presented. It is hoped that they may prove useful in future research into the present problem.

1. It is recommended that research on the same basic problem be continued. Remolded, homionic samples of a highly overconsolidated swelling soil should be used. This allows the many factors affecting swelling to be controlled and evaluated.
2. Investigations of the same soil type using undisturbed samples should also be performed. This would help to substantiate the author's research and indicate the differences between remolded and undisturbed test results.

3. It is recommended that a program be initiated to investigate the differences in free-swell and constant-volume swelling pressures under controlled laboratory conditions.
4. In order to conduct the above proposals, it is recommended that a new method of conducting constant-volume swell tests be designed. Possible suggestions are the use of compressionless consolidation equipment and strain gauges to measure swelling tendencies.

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APPENDIX A

Data Sheets for Soil Classification Tests.

Typical time-consolidation Curves.

APPENDIX A

A.1 Test Information

This appendix includes the laboratory data for the physical classification tests conducted on Bearpaw Shale. Included in these tests are the results of specific gravity, Atterberg limits, and grain size analysis tests.

Typical time-consolidation curves from the consolidation tests performed in this research program are also included. These time-consolidation curves are representative, and were taken from each consolidation test under the maximum consolidation pressure.

UNIVERSITY of ALBERTA
 DEP'T. of CIVIL ENGINEERING
 SOIL MECHANICS LABORATORY
SPECIFIC GRAVITY

PROJECT THESIS
 SITE S.S.R.D.
 SAMPLE BEAR PAW SHALE
 LOCATION
 HOLE DEPTH 108.1-109.1
 TECHNICIAN J. GORIS DATE

Sample No.	1	2	3
Flask No.	J.S.	R.J.R.	P.J.R.
Method of Air Removal	VACUUM	VACUUM	VACUUM
W_{b+w+s} GRAMS	742.09	736.60	701.85
Temperature T °C	26.5	25.5	22.9
W_{b+w}	682.68	683.17	683.49
Evaporating Dish No.			
Wt. Sample Dry + Dish			
Tare Dish			
W_s	93.90	83.87	28.83
G_s	2.72	2.75	2.75

W_{b+w+s} = Weight of flask + water + sample at T°.

W_{b+w} = Weight of flask + water at T° (flask calibration curve).

W_s = Weight of dry soil

G_s = Specific gravity of soil particles = $\frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.	1	2	Sample No.	3
Container No.	1A	2A	Container No.	3A
Wt. Sample Wet + Tare	278.28	213.58	Wt. Test Sample Wet + Tare	200.87
Wt. Sample Dry + Tare			Tare Container	
Wt. Water			Wt. Test Sample Wet	
Tare Container	184.38	189.71	W_s	172.04
Wt. of Dry Soil	93.90	83.87		28.83
Moisture Content w %				

Description of Sample: _____

Remarks: AVERAGE VALUE = 2.74

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DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT THESIS
SITE S.S.R.D.
SAMPLE BEAR PAW SHALE
LOCATION
HOLE DEPTH 108.6-109.1
TECHNICIAN J. GORIS DATE

Liquid Limit

Trial No.	1	2	3	4	5	6
No. of Blows	34	33	31	15	16	14
Container No.	A-16	A-1	A-9	A-17	A-23	A-24
Wt. Sample Wet + Tare	50.02	51.05	53.19	51.38	55.44	56.96
Wt. Sample Dry + Tare	40.38	40.59	41.81	40.01	42.03	42.80
Wt. Water	9.64	10.46	11.38	11.37	13.41	14.16
Tare Container	30.75	29.98	30.31	29.80	30.05	30.29
Wt. of Dry Soil	9.63	10.61	11.50	10.21	11.98	12.51
Moisture Content w%	100.1	98.6	99.0	111.4	111.9	113.2

Average Values

$w_L = 103.8$
 $w_p = 34.5$
 $w_s = 19.8$
 $I_p = 69.3$
 $I_f = 38.7$
 $I_t = 1.79$

Plastic Limit

Trial No.	1	2	3
Container No.	A-52	A-53	A-54
Wt. Sample Wet + Tare	52.506	52.866	55.923
Wt. Sample Dry + Tare	51.137	51.589	53.959
Wt. Water	1.369	1.277	1.964
Tare Container	47.230	47.769	48.375
Wt. of Dry Soil	3.907	3.820	5.584
Moisture Content %	35.04	33.43	35.17

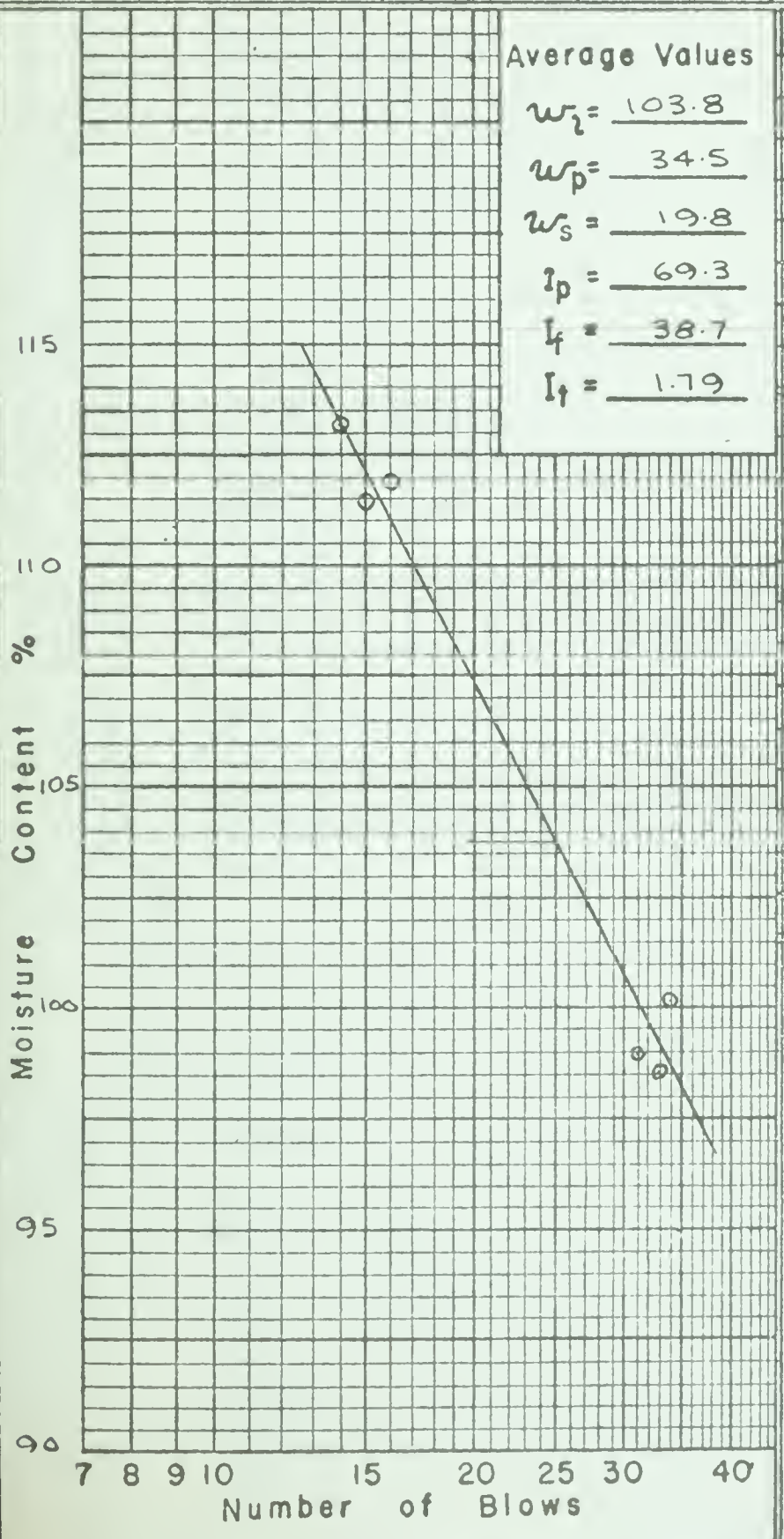
Shrinkage Limit

Trial No.	1	2	3
Container No.	93	97	87
Wt. Sample Wet + Tare	139.86	136.97	133.90
Wt. Sample Dry + Tare	130.41	127.37	124.06
Wt. Water	9.45	9.60	9.84
Tare Container	124.80	122.30	116.12
Wt. of Dry Soil W_o	5.61	5.07	7.94
Moisture Content $w\%$	168.4	189.3	123.9
Vol. Container V	12.51	12.41	12.24
Vol. Dry Soil Pat V_o	3.80	3.83	3.97
Shrinkage Vol. $V - V_o$	8.35	8.58	8.27
Shrinkage Limit w_s	19.6	20.1	19.8

$$w_s = w \left(\frac{V - V_o}{W_o} \times 100 \right)$$

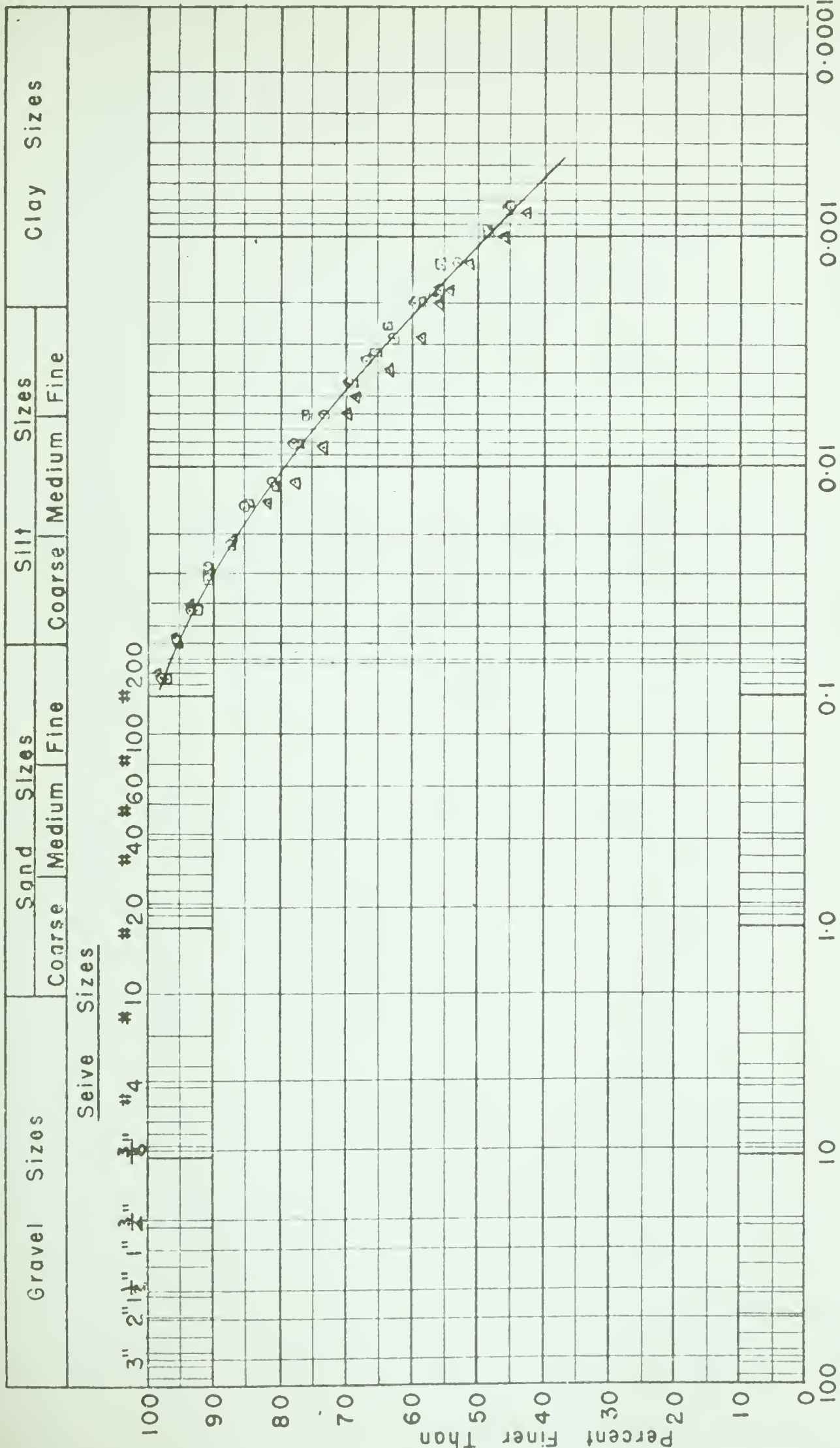
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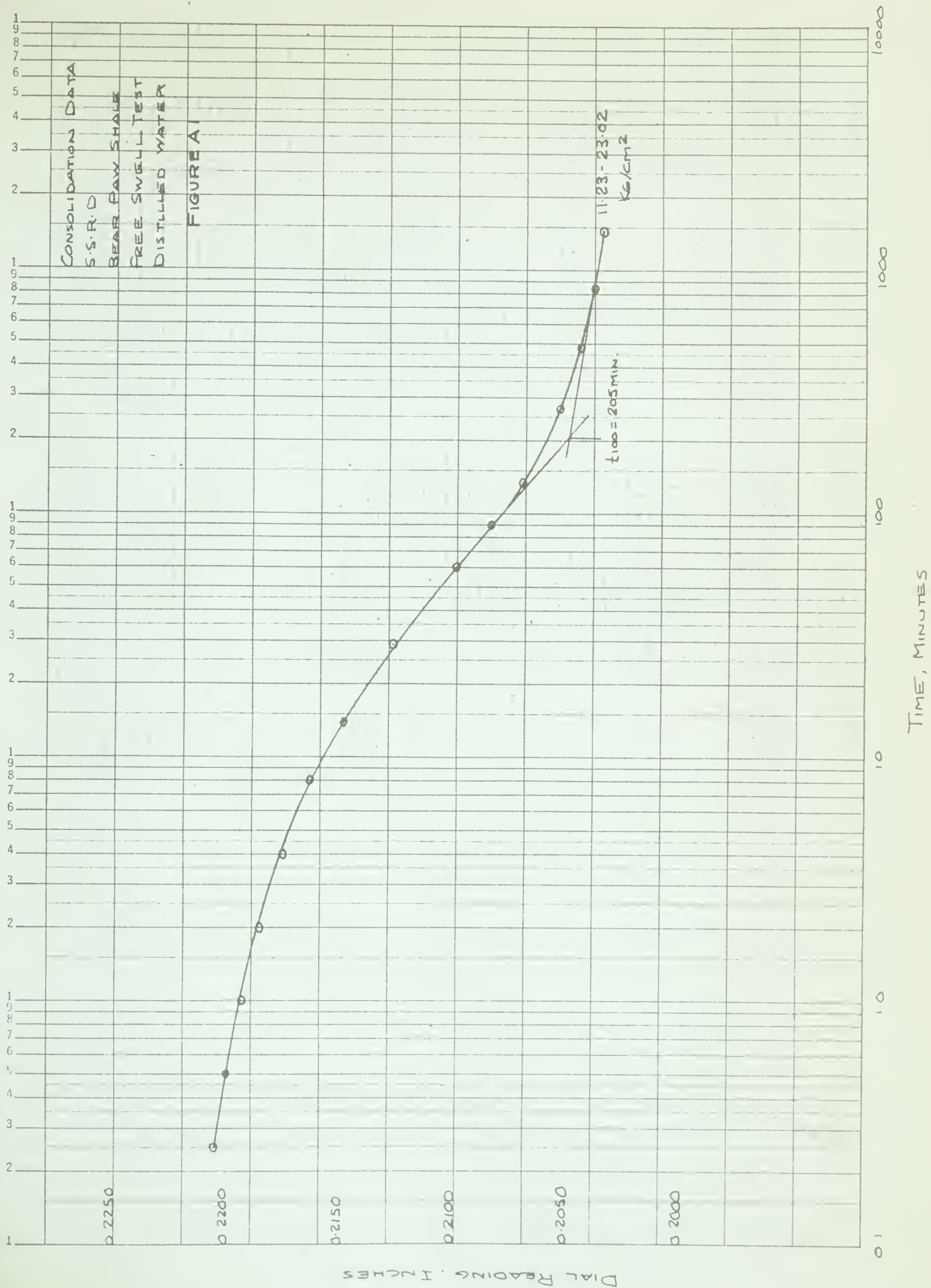
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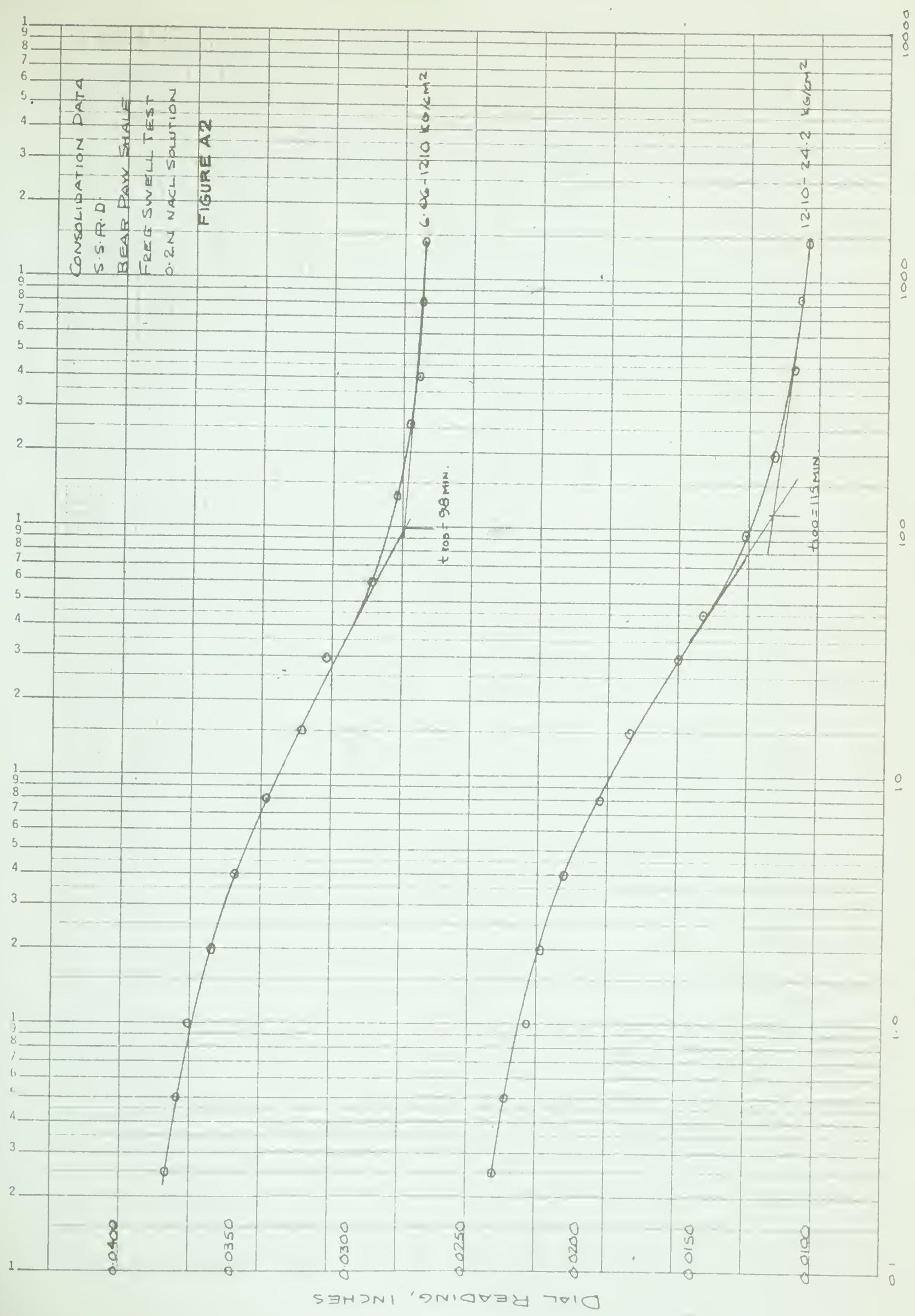


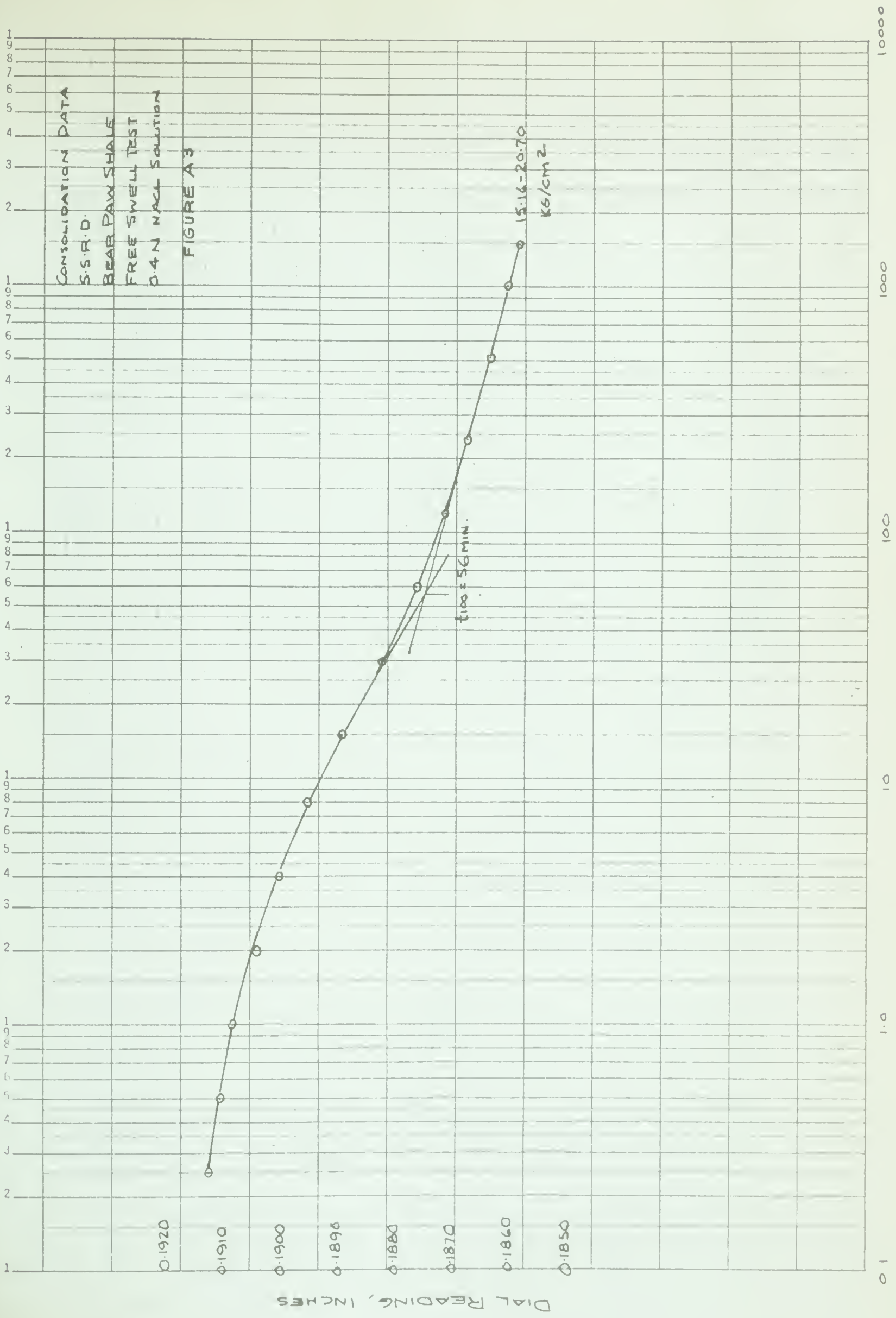
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DEPT of CIVIL ENGINEERING
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GRAIN SIZE CURVE

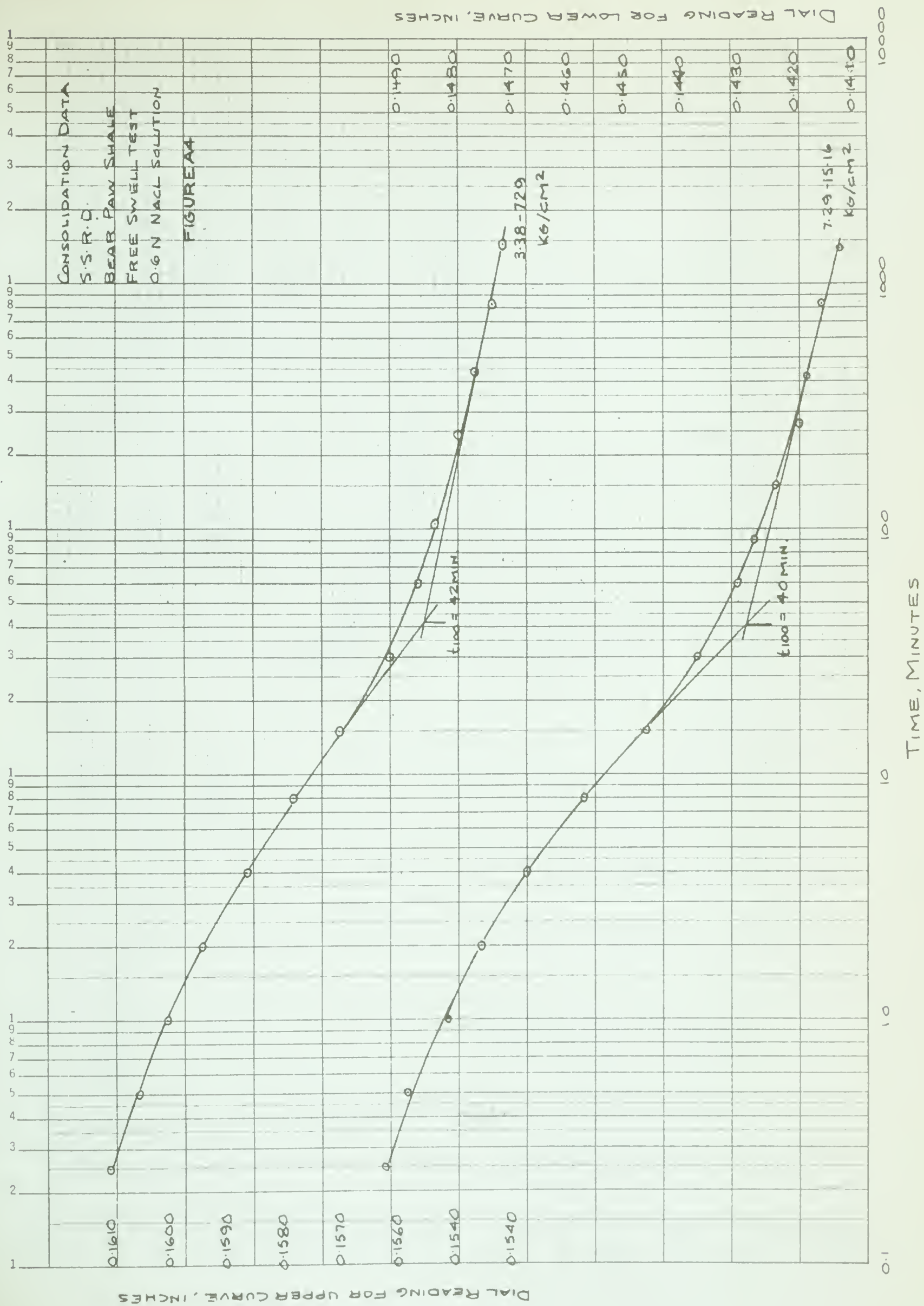
PROJECT Thesis
SITE S.S.R.D.
SAMPLE Beauregard #10
LOCATION R.D. 3754 #44C
HOLE _____ DEPTH 108.6-109.1
TECHNICIAN J. Goris DATE 24 Jan 65

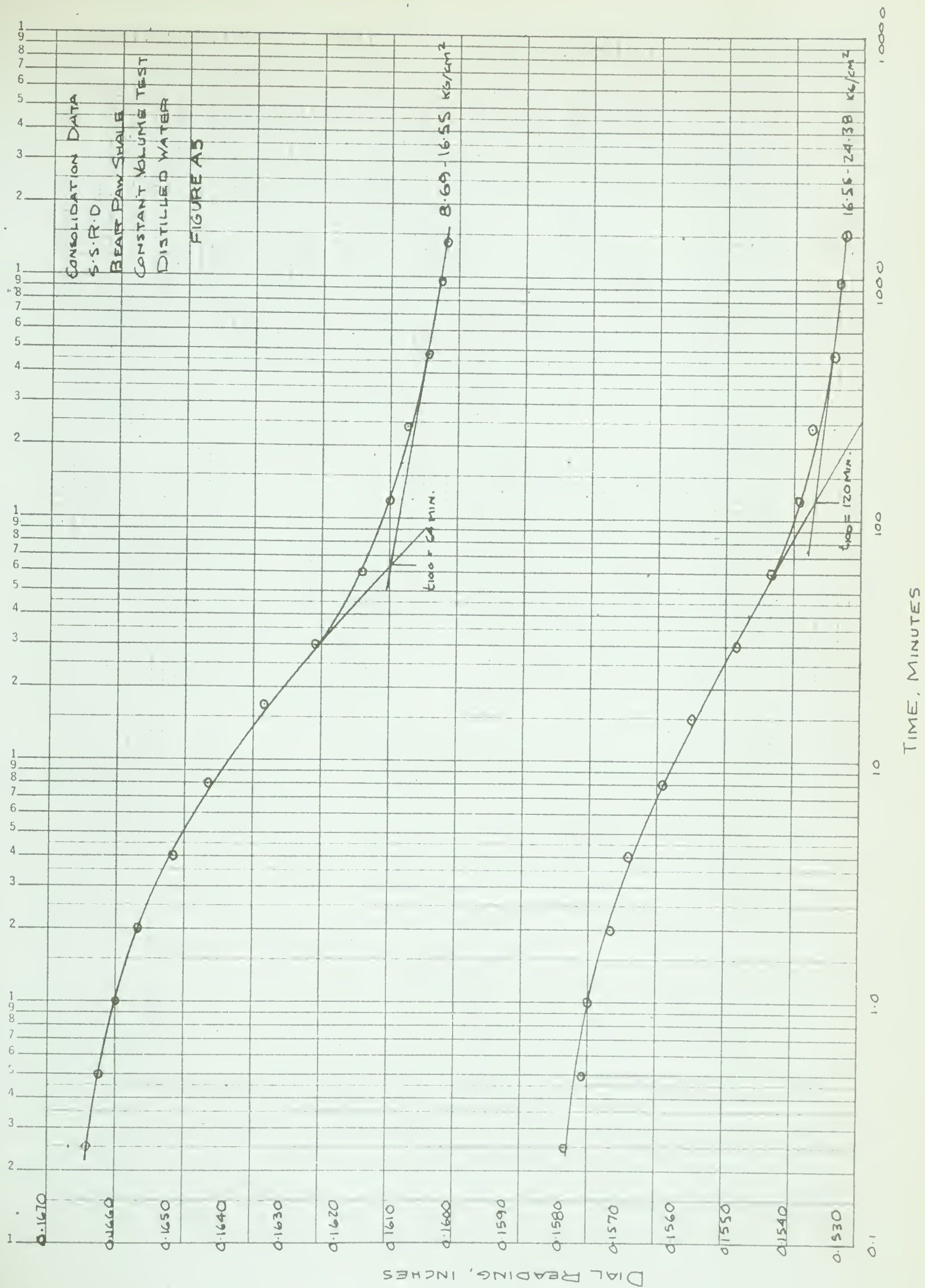


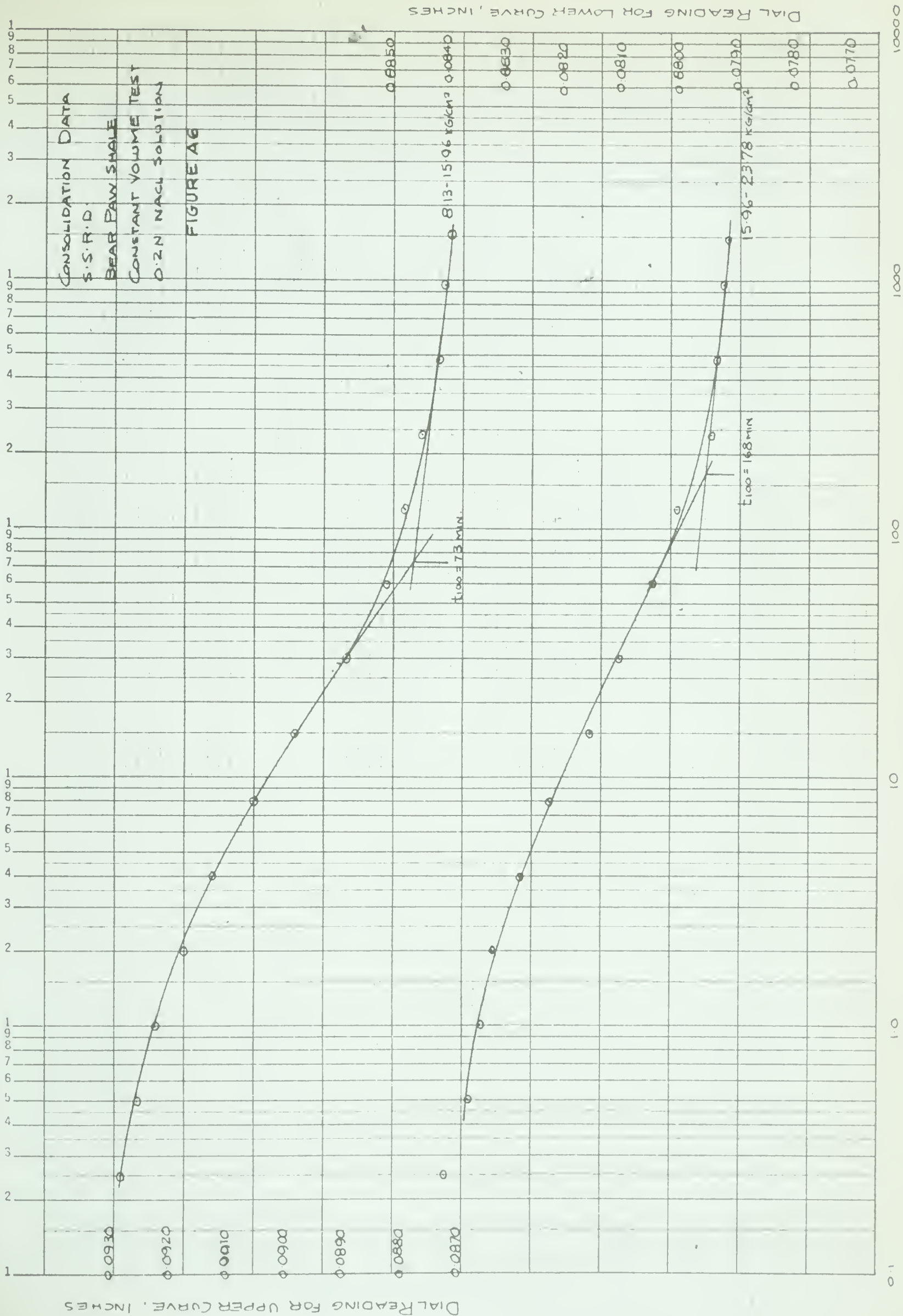


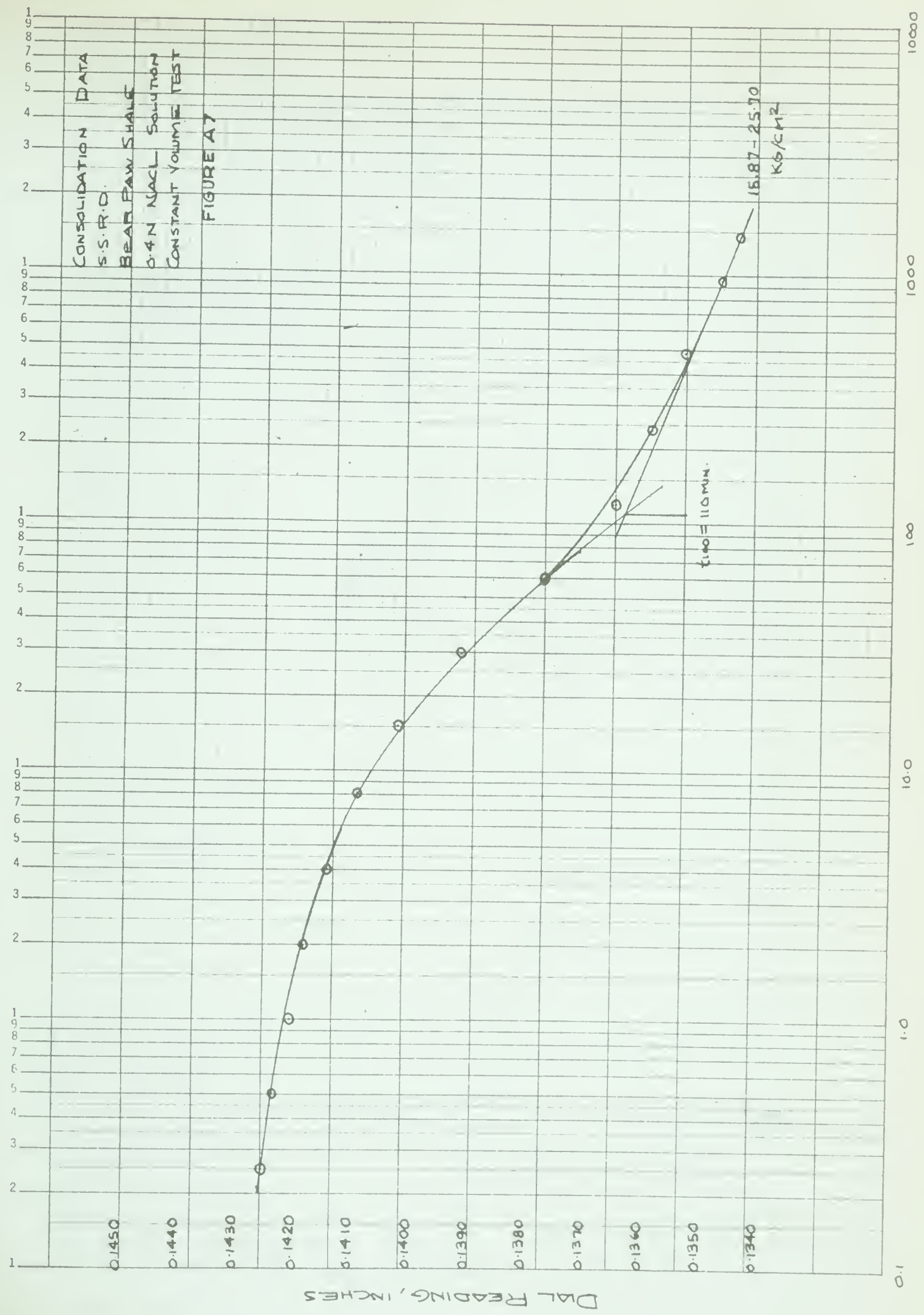


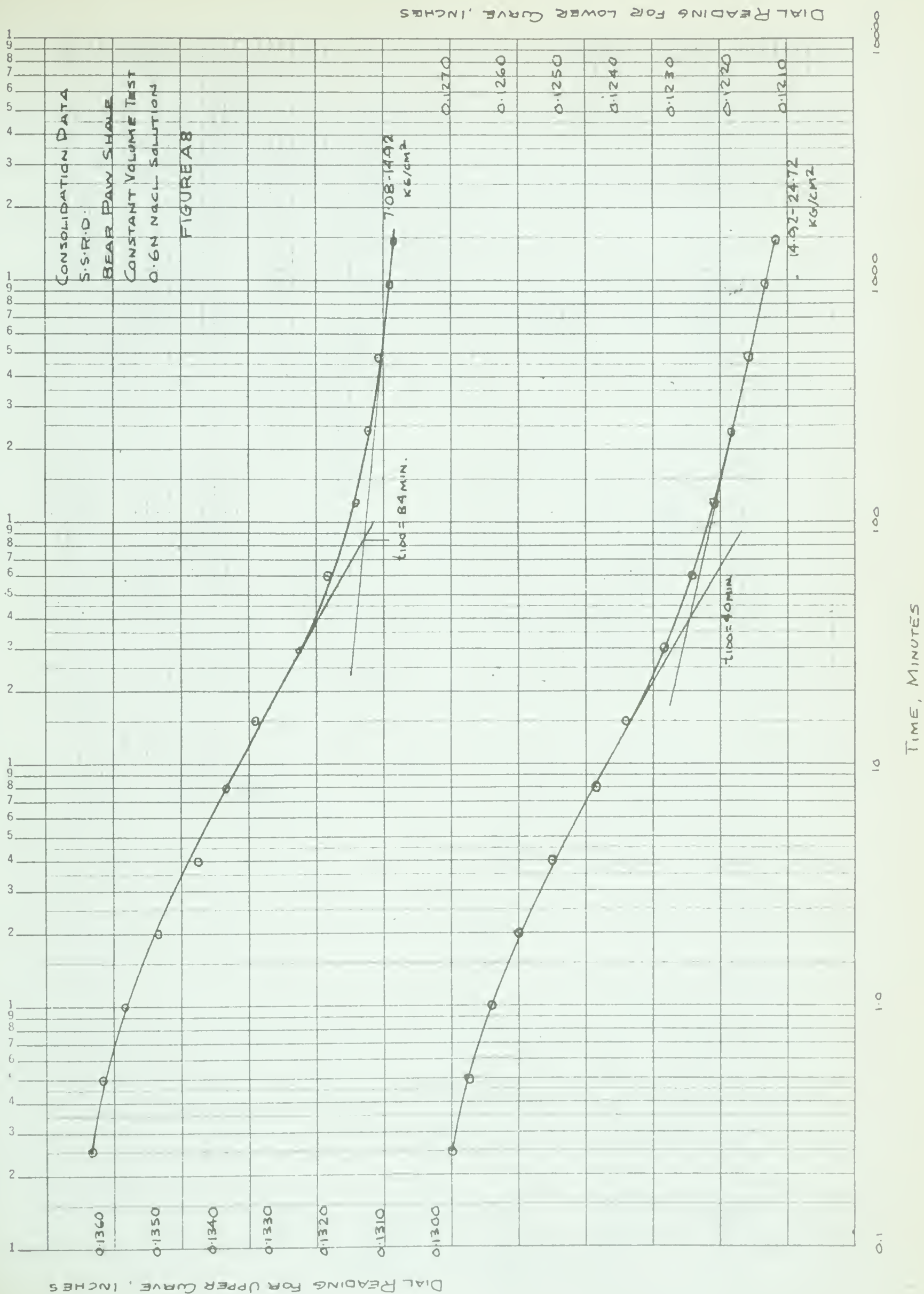


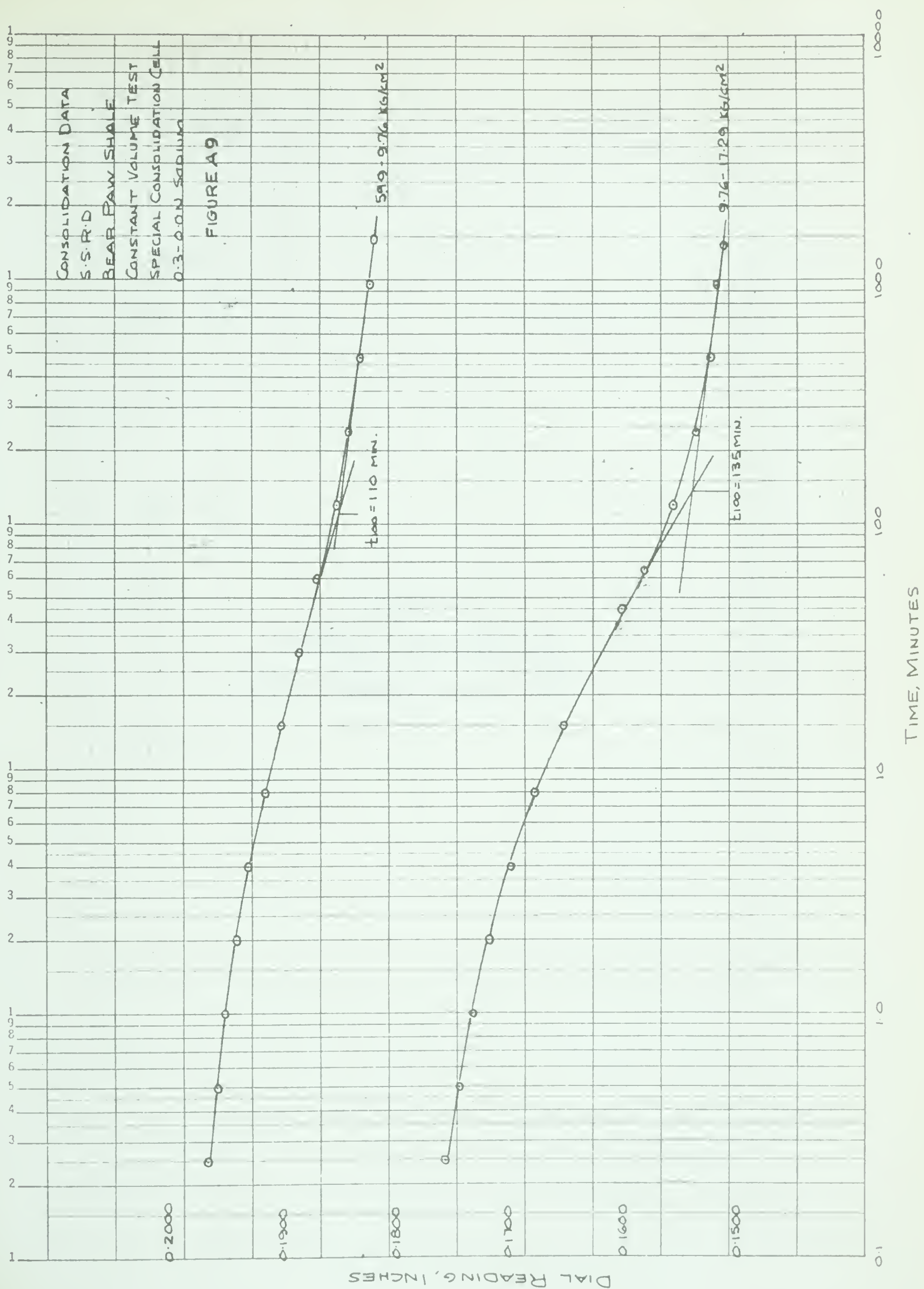












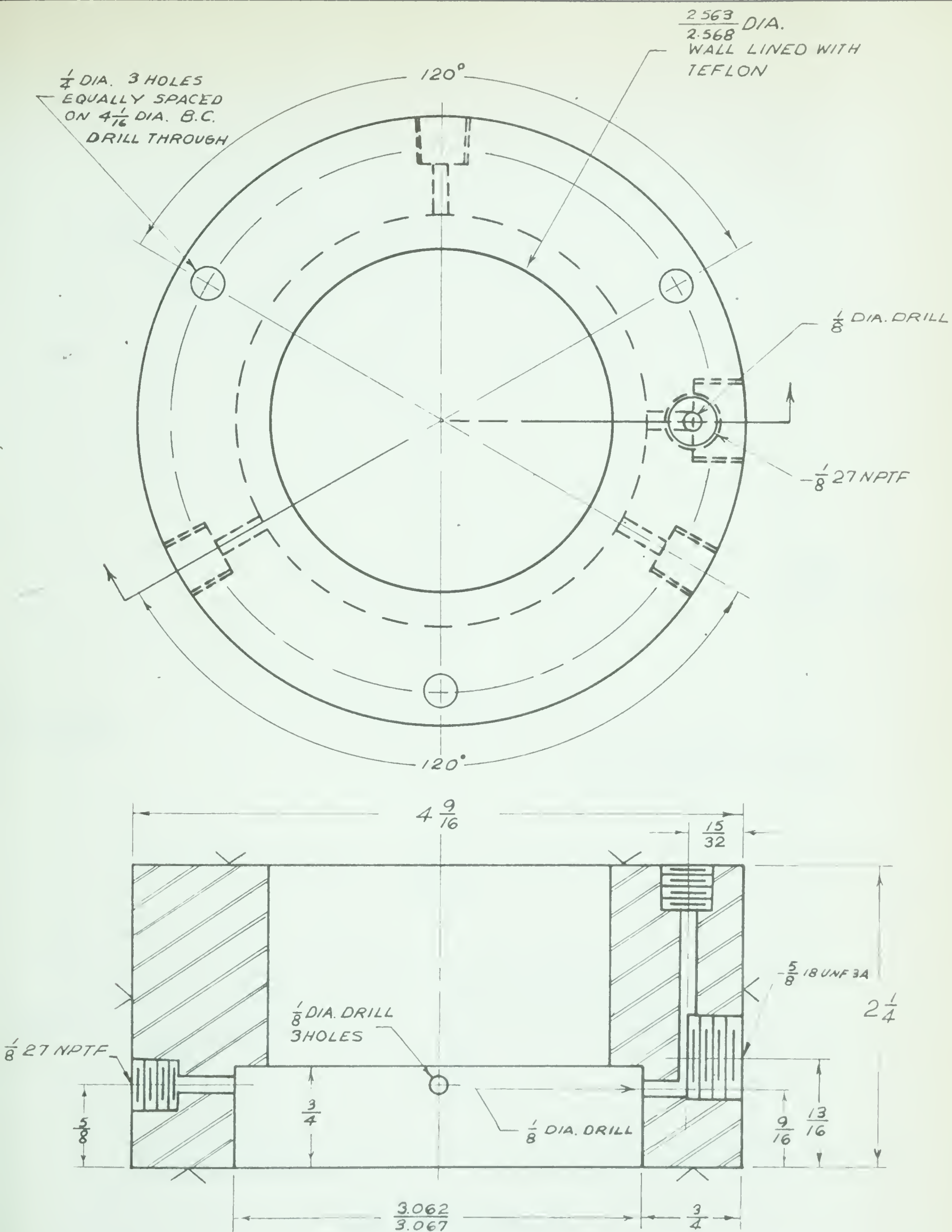
APPENDIX B

Machine Drawings for
Special Consolidation Cell

APPENDIX B

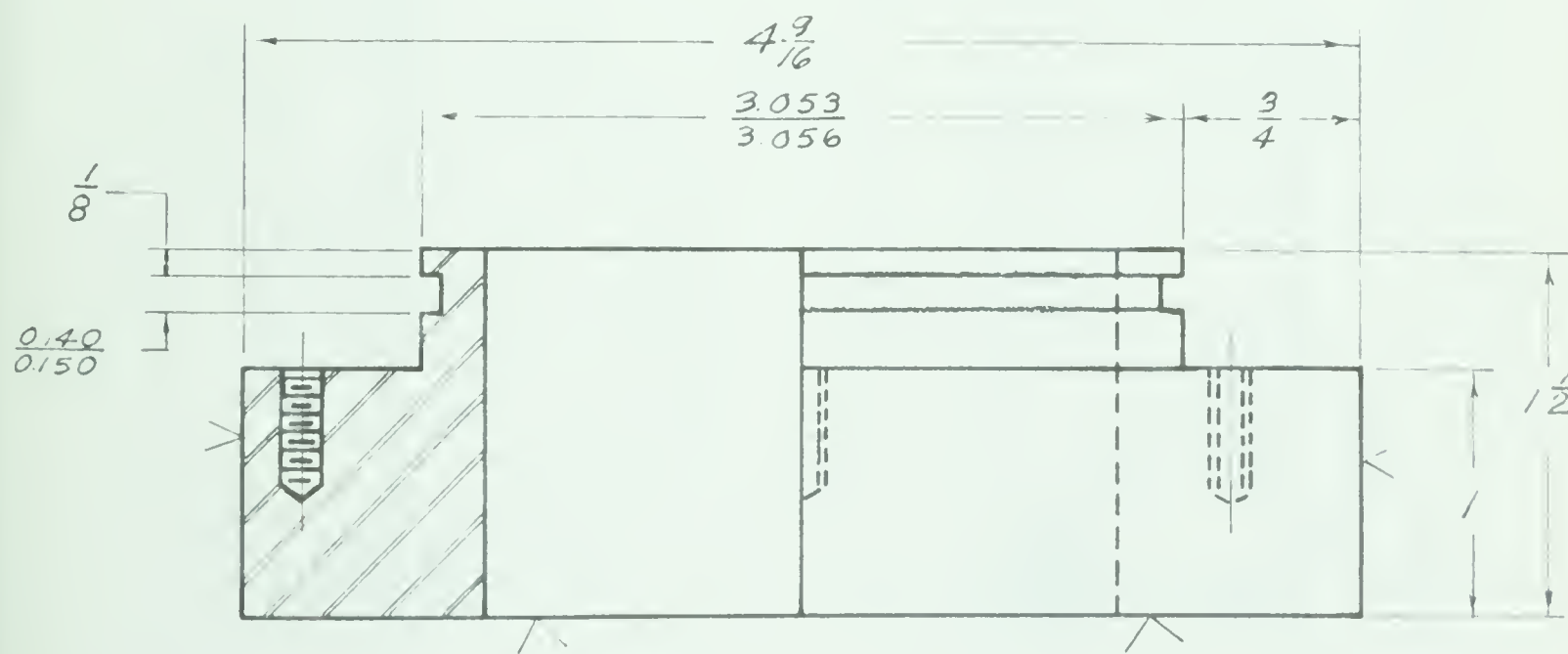
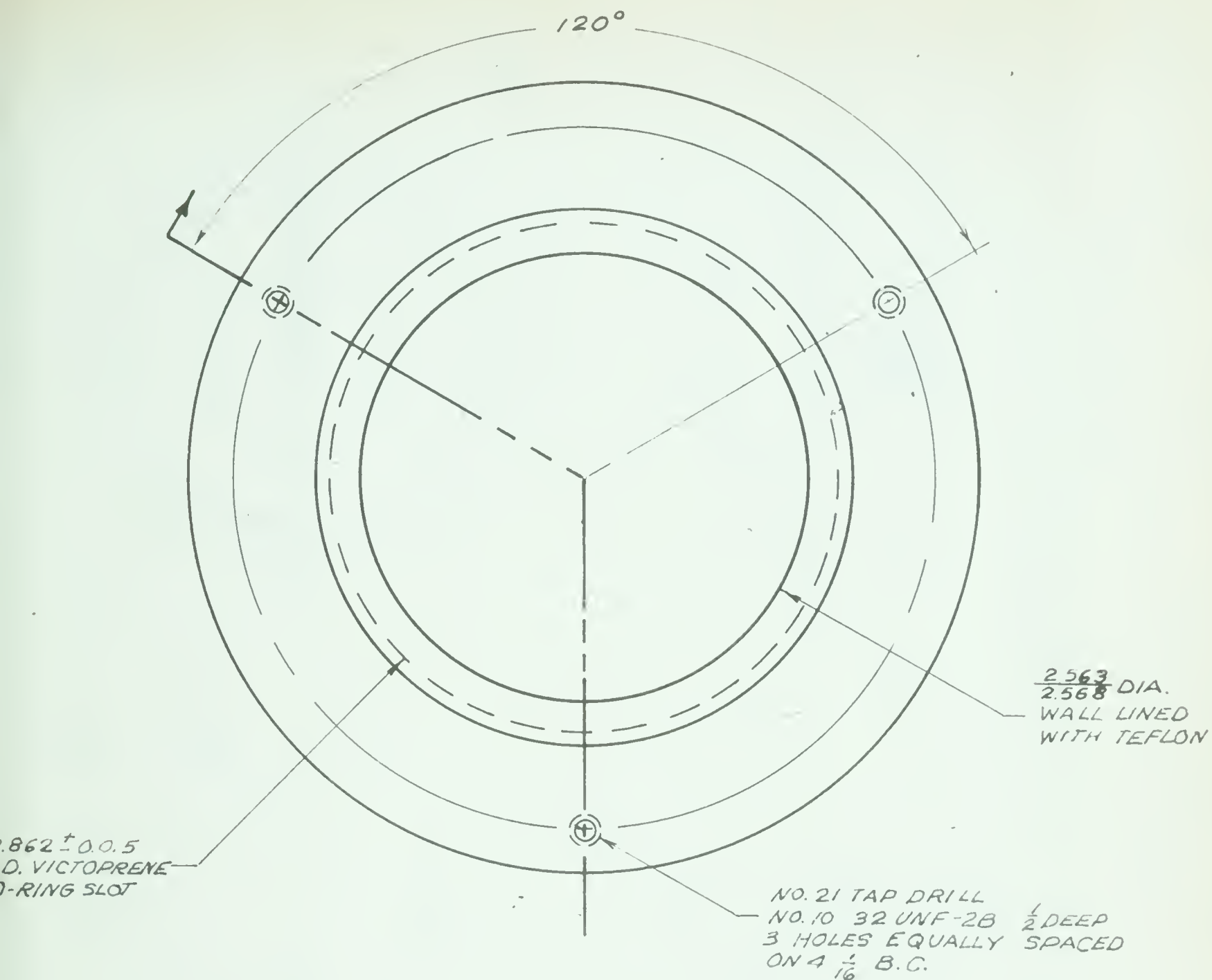
B.1 General

Chapter II contains a brief description of the special consolidation cell. An assembly drawing was presented in FIGURE 1 showing the component parts of the cell. This appendix contains detailed machine drawings from which the special consolidation cell was constructed. The dimensions given in these drawings are all in inches. The scale is full. The complete cell, including the upper and lower pistons was constructed of stainless steel.



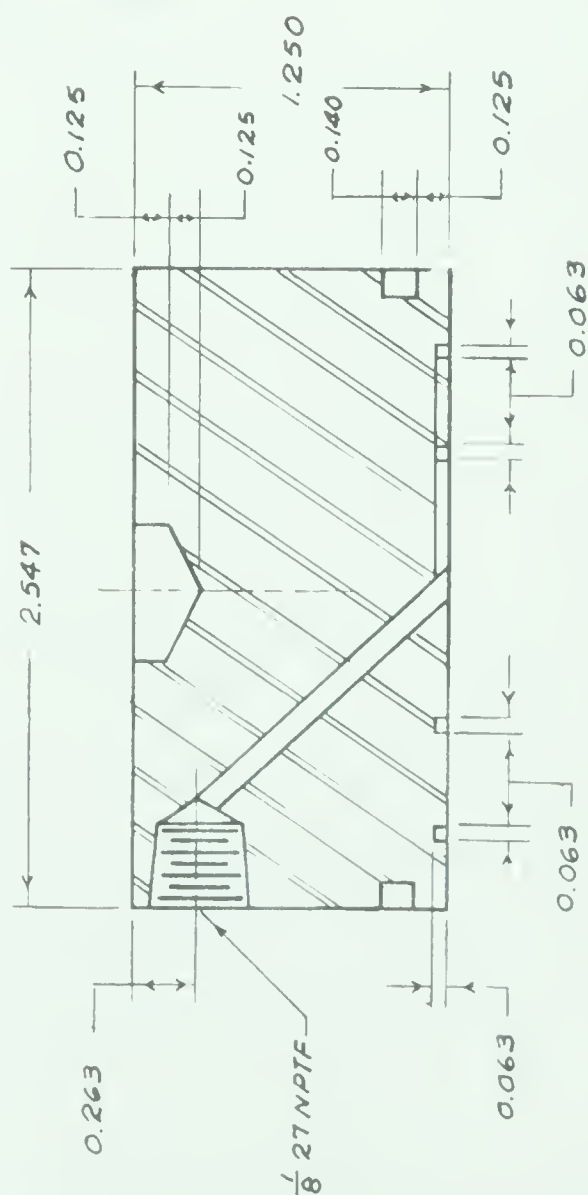
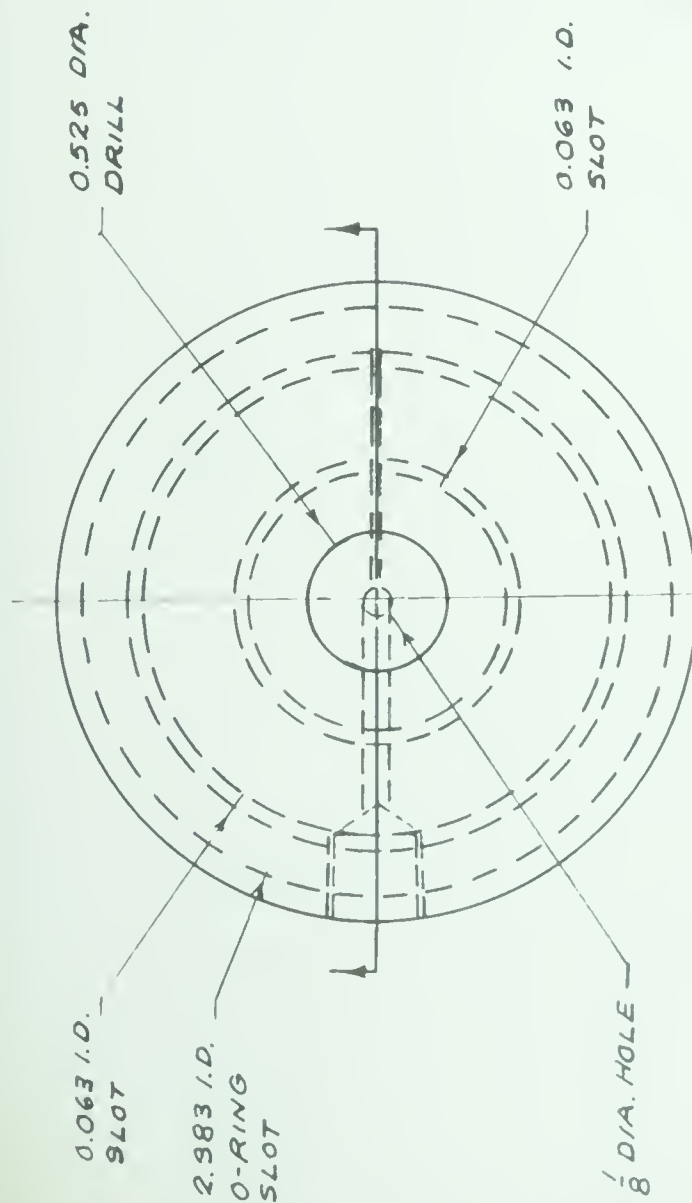
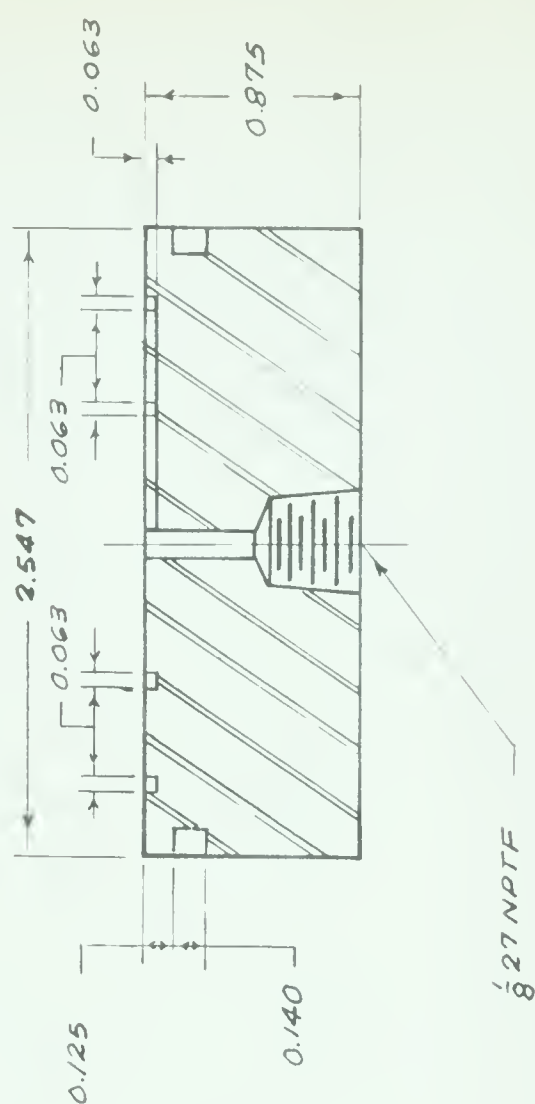
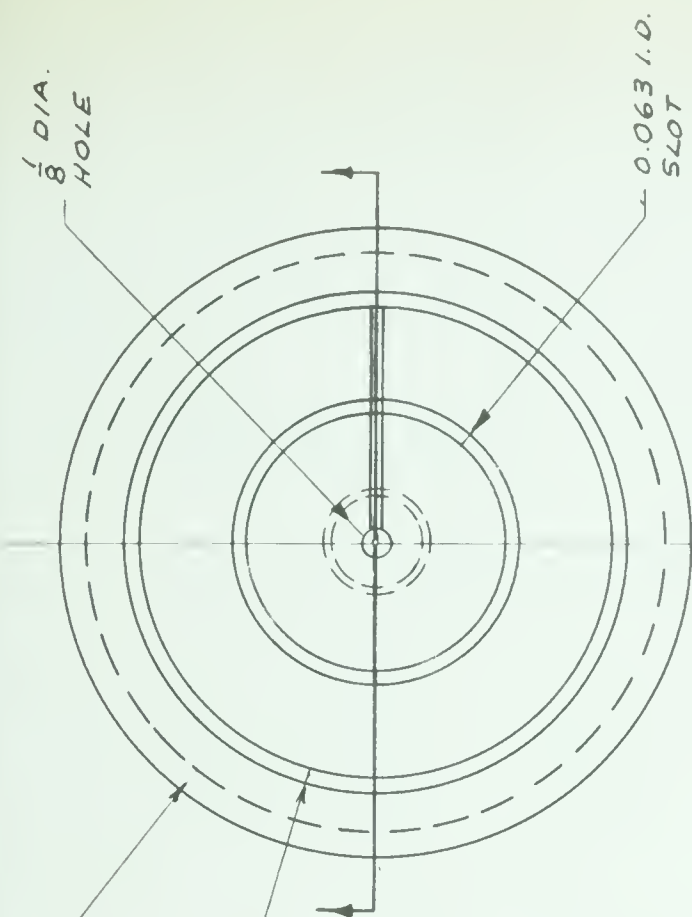
TOP PLATE

SCALE: FULL
 MATERIAL: STAINLESS STEEL
 DRAWN BY: J. GORIS



BOTTOM PLATE

SCALE: FULL
 MATERIAL: STAINLESS STEEL
 DRAWN BY: J. GORIS



TOP PISTON

BOTTOM PISTON

UNIVERSITY OF ALBERTA
DEPT OF
CIVIL ENGINEERING

CELL
PISTONS

MATERIAL: STAINLESS STEEL
SCALE: FULL
DATE: 10-5-64

APPENDIX C

Calibration Data

APPENDIX C

C.1 General

In Chapter II a discussion of the special laboratory test equipment was given. This equipment included a pressure transducer and the special consolidation cell. The use of this equipment for investigating swelling properties of the Bearpaw Shale required calibration information and compressibility studies. This information was necessary before the test results could be properly interpreted.

This appendix includes the calibration curves for the pressure transducer employed in this study to measure pore-water pressures. The compressibility-pressure curves for the special consolidation cell and for the Wykeham-Farrance consolidation apparatus are also included.

C.2 Pressure Transducer and Calibration Curves

Detailed performance and electrical information concerning the pressure transducer are included in this appendix. Information concerning the specifications for the transducer, including its

pressure ranges, mounting and handling instructions, and an orthographic drawing are also included.

The calibration of the pressure transducer was carried out in the special consolidation cell. A Baldwin strain gauge indicator was used to measure strain taking place within the transducer as a fluid pressure was applied. Pressures ranging from atmospheric to 93.00 psi were applied to the transducer in 16 load increments. Strain readings were taken after each load increment.

The process of loading was carried out four separate times. The values of strain readings were averaged and a calibration curve for the loading process was plotted. This curve is shown in FIGURE C1.

Two additional calibration curves for the same pressure transducer are shown in FIGURE C1. These calibration curves were determined from two separate calibrations performed independently of the special consolidation cell (Hollingshead, 1965). One curve was determined by calibrating the transducer against pressure applied by means of water in a triaxial consolidation cell. A Bourdon pressure gauge was used to measure the applied pressure. The remaining curve was developed by calibrating the transducer against a dead load. It is noted that these curves are very

similar except in the lower pressure ranges. The curve determined by the special consolidation cell deviates from that determined in the triaxial consolidation cell by + 9.2% at the maximum deviation.

C.3 Compressibility of the Special Consolidation Cell

The procedure for obtaining the compressibility curve for the special consolidation cell was given in section 2.6 of Chapter II. The results of the compressibility study are presented in this appendix.

FIGURE C2 shows the results of a compression-time-pressure study conducted on various components of the special consolidation cell and the Wykeham-Farrance Bench Model No. 3. In this study, various loads were applied to the testing apparatus for given time intervals. The compression taking place during these time intervals was recorded and a family of compression-time curves was plotted. It is noted that the curves for the pressures applied are straight lines. The data obtained from this study were scattered and were consequently averaged to produce the linear relationships. The time period involved for each pressure was 24 hours.

Before applying these results to future research, it is advisable to conduct a similar study for any particular pressure. This is due to a hysteresis

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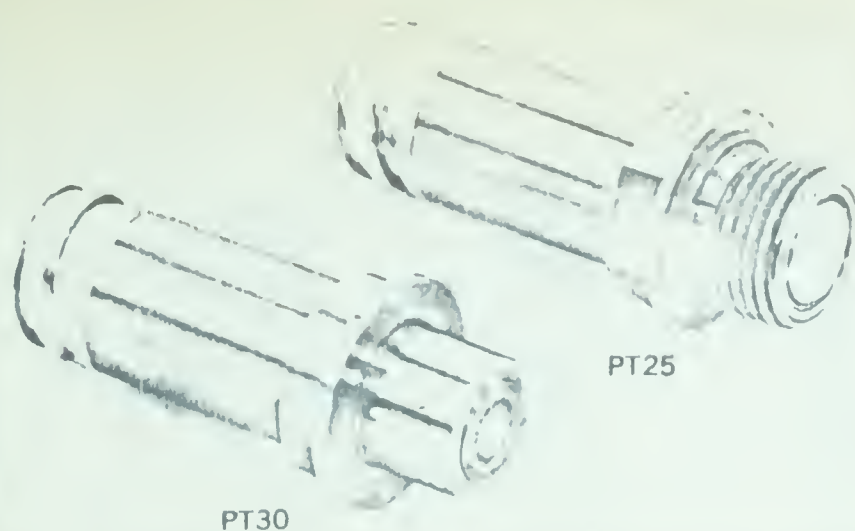
Before applying these results to future research, it is advisable to conduct a similar study for any particular pressure. This is due to a hysteresis

effect developed in the test apparatus from loading and unloading cycles.

FIGURE C3 presents a calibration curve of compression vs. pressure for a 24-hour time period. This curve was plotted with the aid of the compression-time-pressure curves. The apparatus was loaded and unloaded rapidly so that no time effects could take place. Compression was recorded almost instantaneously. After the loading and unloading had taken place, corrections from the time-compression curve of FIGURE C2 were applied for the 24-hour period of loading. This resulted in the compression-pressure curve shown in FIGURE C3.

C.4 Compressibility of the Wykeham-Farrance Consolidation Apparatus

FIGURE C4 presents the results of a compressibility study performed by Fredlund (1964). The results of this study were used in conjunction with the swelling pressure investigation conducted by the author on Bearpaw Shale. Compressibility corrections were applied to the results of the free-swell and constant-volume swell tests performed in the Wykeham-Farrance consolidation apparatus. The compressibility curve is included in this appendix for future research.



REPLACES BULLETINS
NO. 225 and 230

BULLETIN NO. 225A

DECEMBER 1961
Pressure Transducers
Model PT25 and PT30

PRESSURE TRANSDUCERS • Model PT25 Model PT30

These models incorporate a type of case construction which effectively isolates the strain gage mechanism from mechanical shock and external thermal transients. The isolation between the outer case and the sensing element reduces to a negligible amount the effect upon the strain gage of mechanical distortion of the case attendant with the threaded mounting of the PT25 and with the clamp or ring mounting of the PT30.

By virtue of the flush diaphragm design of these transducers, optimum frequency response can be realized in dynamic measurement applications.

Electrically, the PT25 and the PT30 incorporate a miniature Dynisco developed unbonded strain gage in the configuration of a four active-arm resistive Wheatstone Bridge. This precision gage is responsible for the inherent accuracy, stability, and performance dependability of the transducers.

STANDARD MODEL PT25 AND PT30 TRANSFORMER MODEL PT25-100-1000

PERFORMANCE

Maximum Non-linearity	0.5% F.S.
Maximum Hysteresis	0.25% F.S.
Repeatability	0.1% F.S.
Temperature Range	-65 to +300°F.
Temperature Drift*	2% F.S./100°F.
Temperature Effect on Sensitivity*	1% F.S./100°F.
Vibration Effect	0.007 psi/'G'
Rated Mounting Torque (PT25)	100 inch-pounds

*Specification represents a standard deviation of test sampling

ELECTRICAL

Configuration	Four active-arm Wheatstone Bridge
Bridge Resistance	350 ohms nominal
Full Scale Output	4.00 mv/v (minimum)
Zero Pressure Output	±1.00 mv/v
Excitation Voltage	6 volts ac or dc



DYNISCO

DIVISION OF AMERICAN BRAKE SHOE COMPANY

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SPECIFICATIONS FOR PRESSURE TRANSDUCER MODELS PT25 and PT30

PRESSURE RANGES

Model Number		Pressure Range (psi)	Accuracy (%)	Operating Temp. Range (°F)
PT25-10	PT30-10	0-10	3	5,000
PT25-30	PT30-30	0-30	3	6,000
PT25-50	PT30-50	0-50	2	8,000
A PT25-1C	PT30-1C	0-100	2	10,000
PT25-3C	PT30-3C	0-300	2	11,000
PT25-5C	PT30-5C	0-500	2	11,000
PT25-1M	PT30-1M	0-1000	1.5	12,000
PT25-3M	PT30-3M	0-3000	1.2	12,000
PT25-5M	PT30-5M	0-5000	1.2	12,000

PT25 MOUNTING AND HANDLING INSTRUCTIONS

Every PT25 is supplied with a metal backed rubber washer which will provide a pressure tight seal when clamped between the body of the transducer and a flat mounting surface. A mounting torque of approximately 100 inch-pounds is recommended. Always use a torque wrench to insure that the maximum torque rating of 150 inch-pounds is not exceeded or a permanent shift in the transducer's calibration may result.

A protective cap covering the diaphragm is supplied with every transducer. It is suggested that this cap be kept on the gage whenever it is not in actual service.

PT30 MOUNTING AND HANDLING INSTRUCTIONS

Every PT30 is supplied with a metal backed rubber washer which will provide a pressure tight seal when clamped between the body of the transducer and a flat mounting surface having a $\frac{1}{16}$ inch diameter orifice. Hold down clamps are not provided with this model except upon special order.

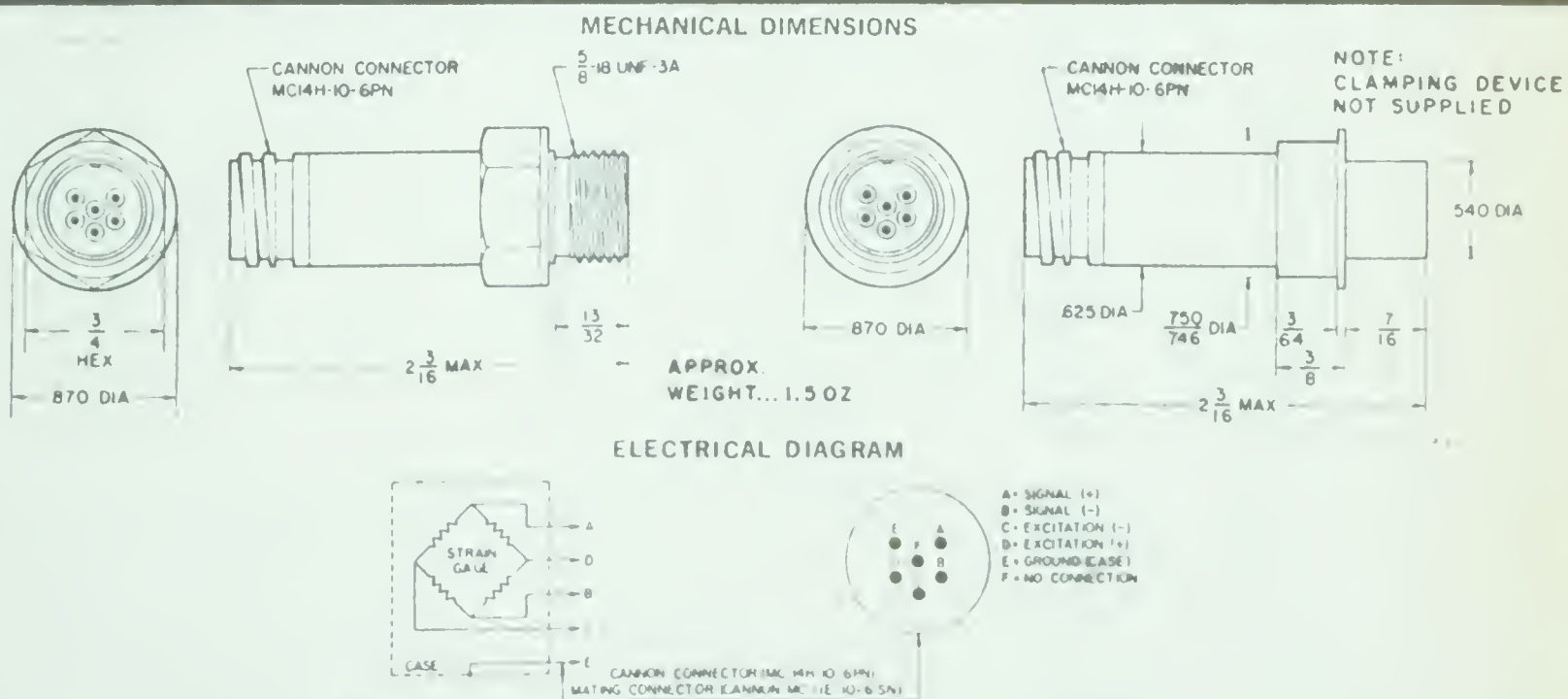
For general safety and ease of mounting, the Model PT25 is recommended for applications over 500 psi.

ORDERING INFORMATION

The model number as given in the above table designates a gage pressure unit. If an absolute pressure unit is desired, an 'A' is placed before the model number, e.g., APT25-1C. Temperature calibrated units are ordered by placing 'TC' before the model number, e.g., TCPT25-1C or TCAPT25-1C. These units are calibrated at various temperatures throughout their rated operating range and are compensated electrically, if necessary, to guarantee conformity to the above performance specifications. Transducers can be ordered in any intermediate pressure range not listed in the above table at no additional cost.

TERMS: Net 30 days

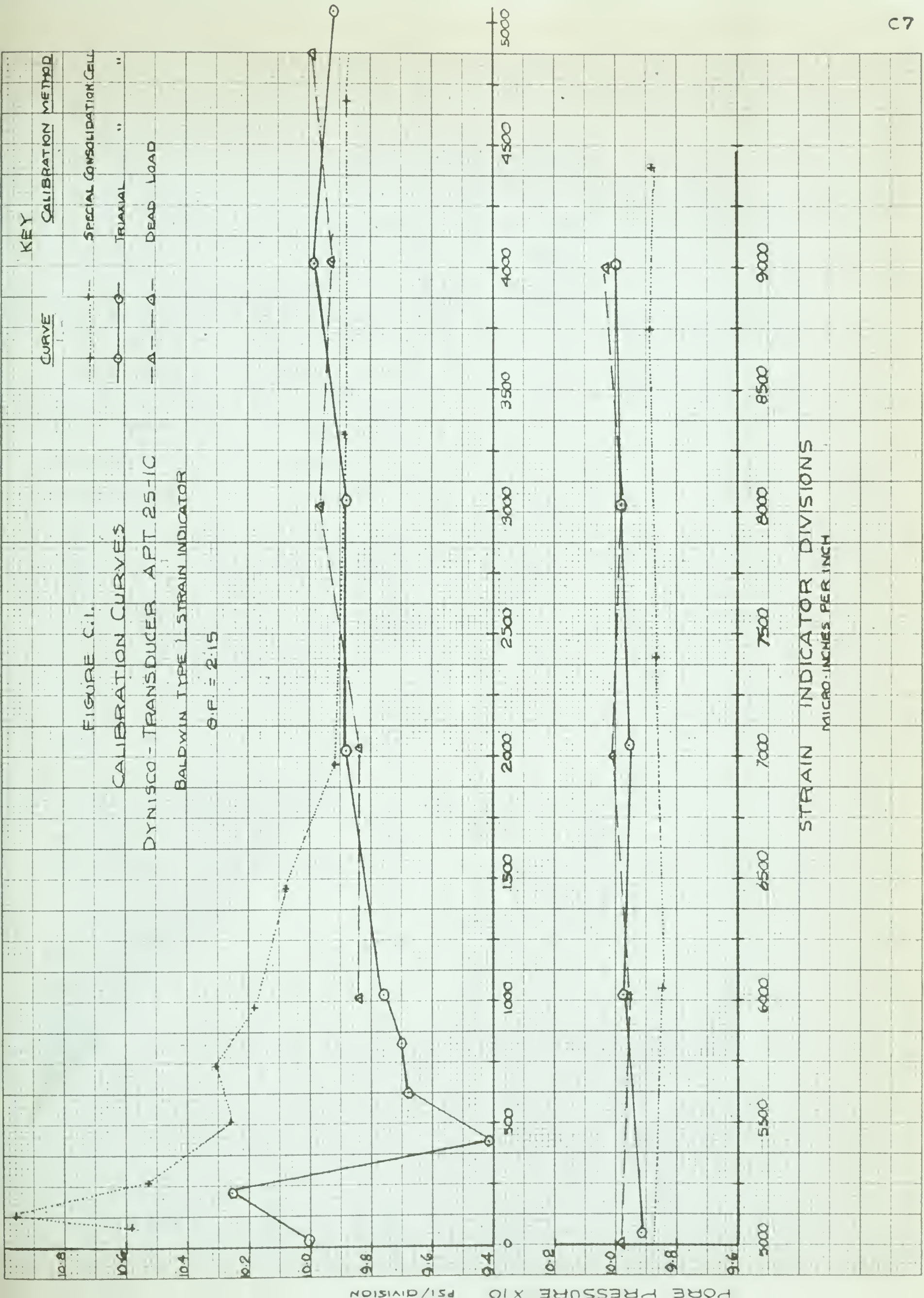
F.O.B. Cambridge, Mass.

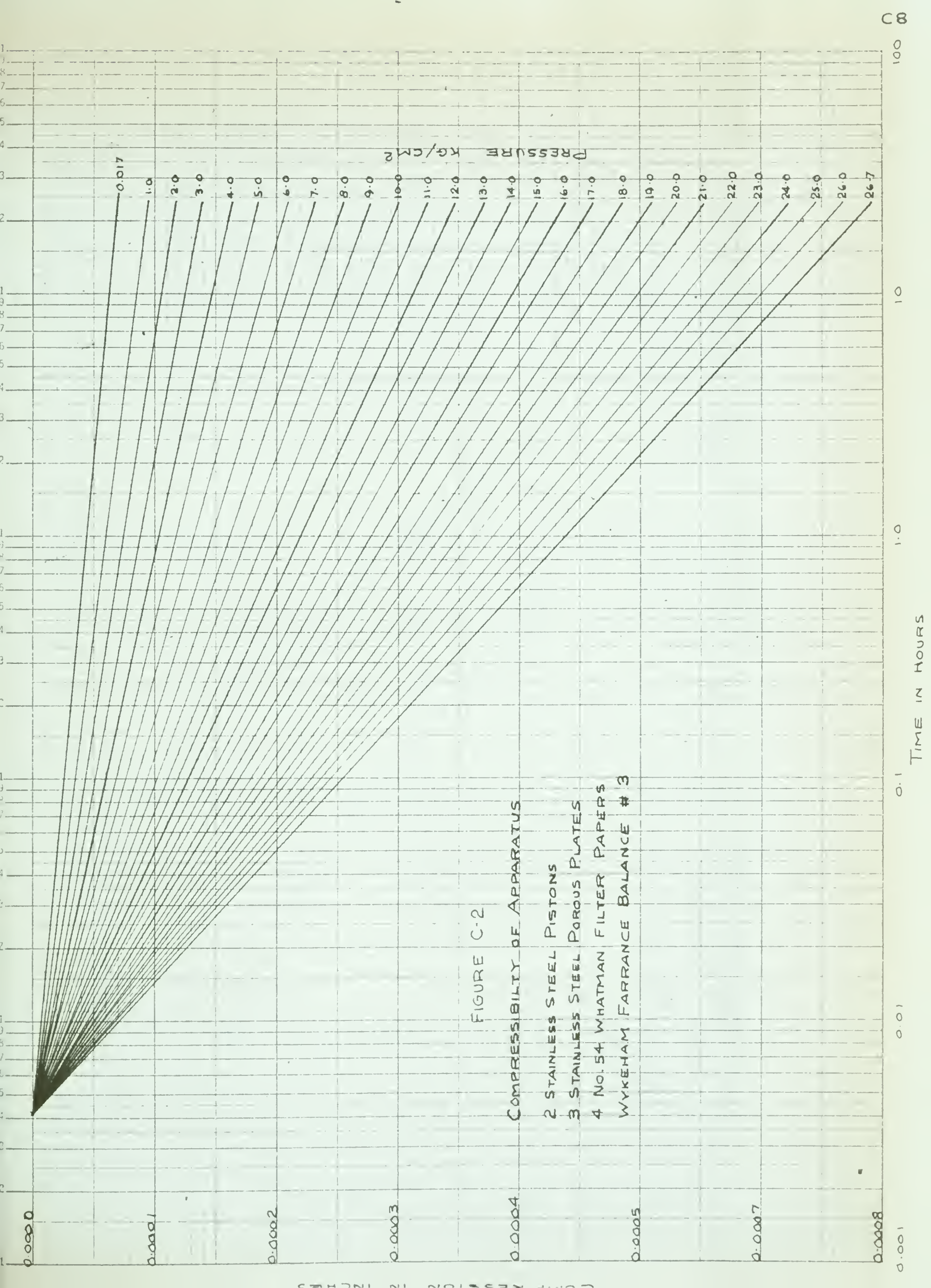


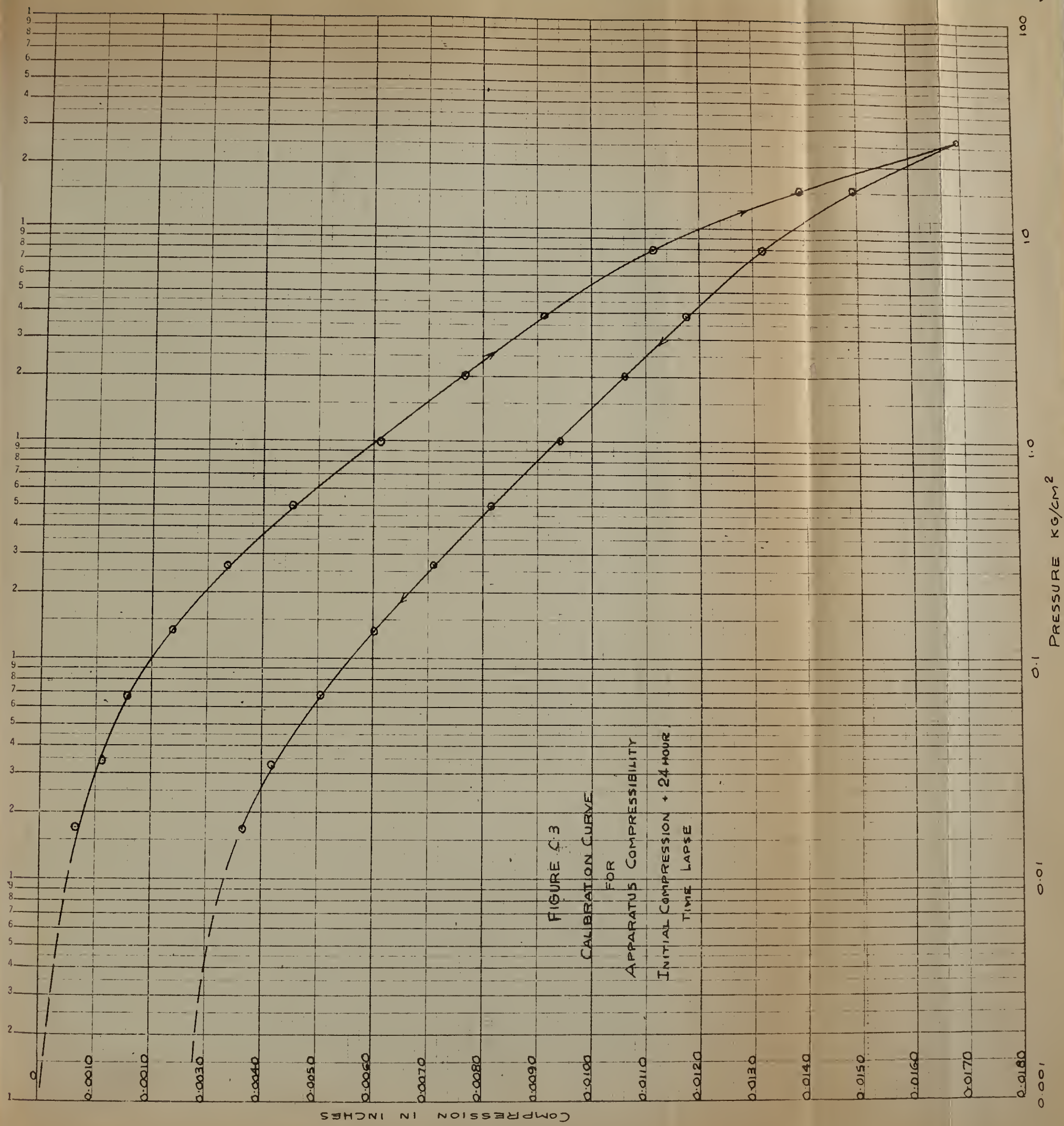
DYNISCO

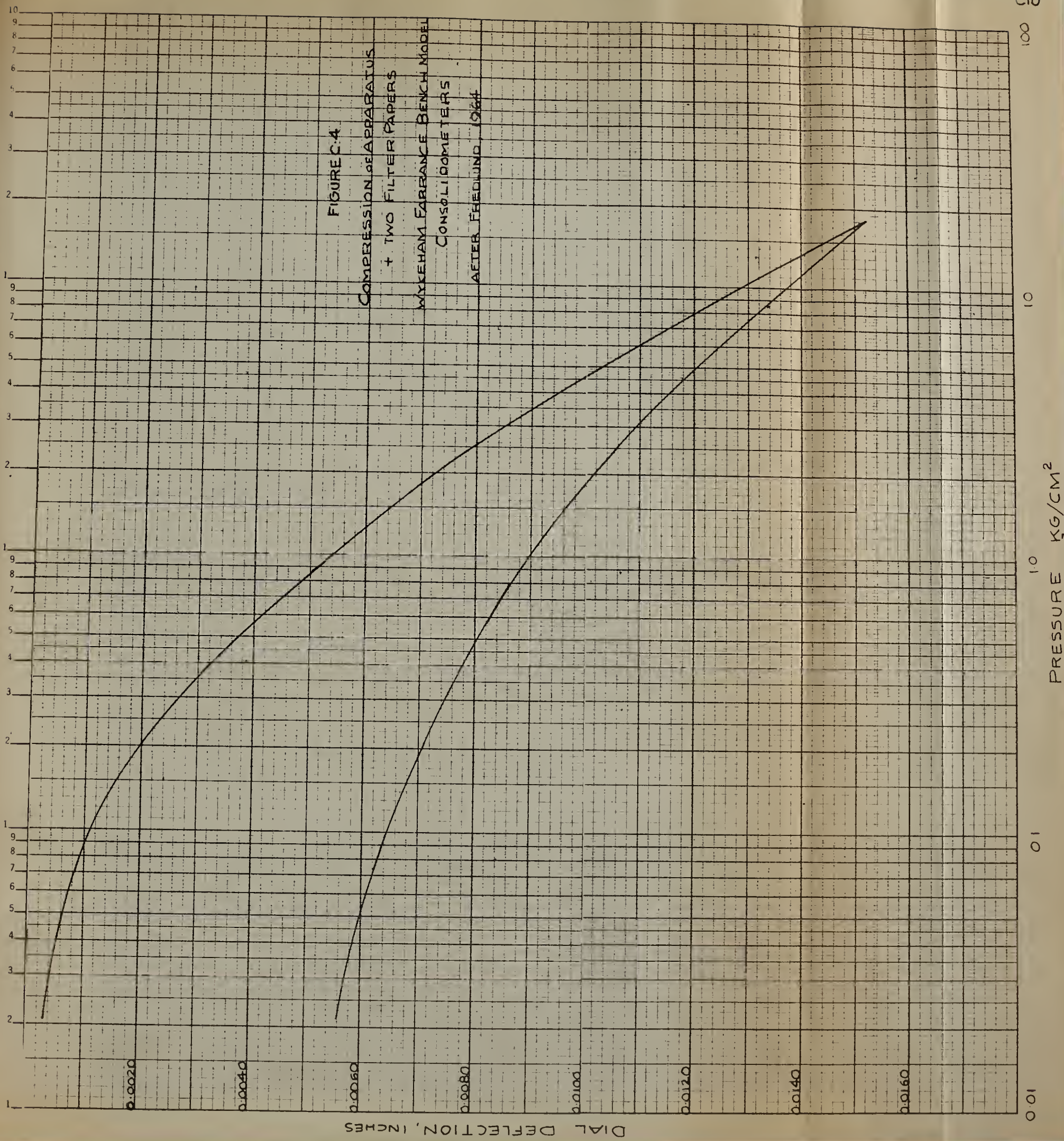
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APPENDIX D

Pore Pressure Reaction Test Results

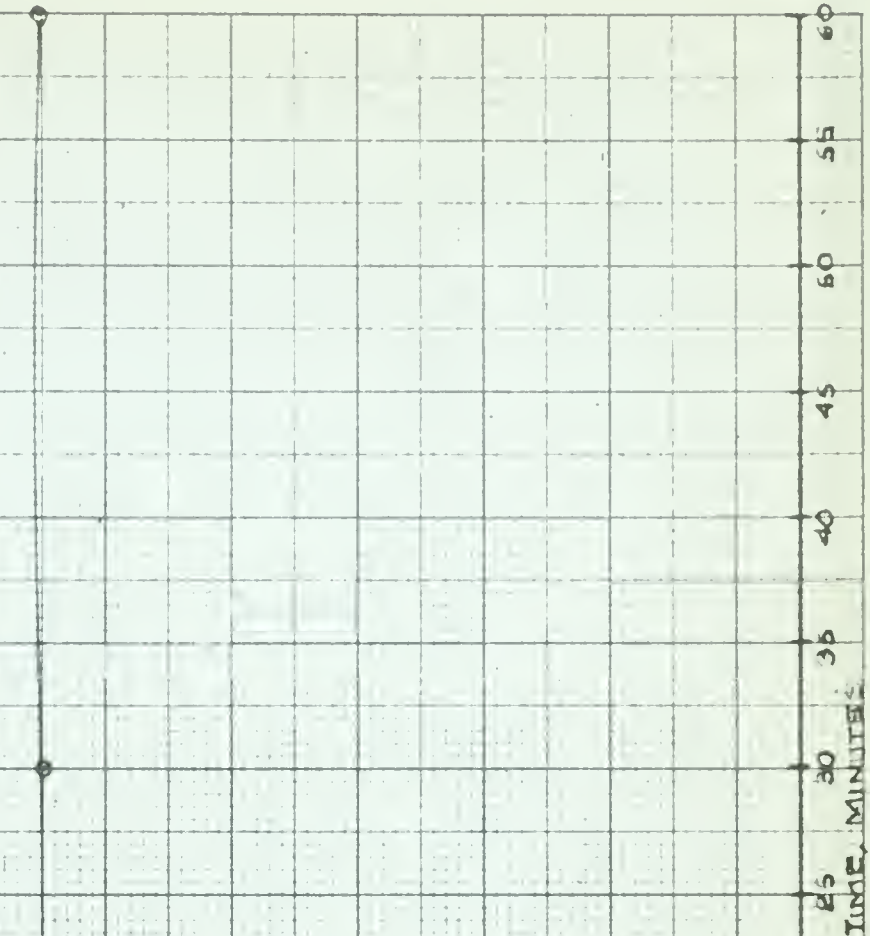
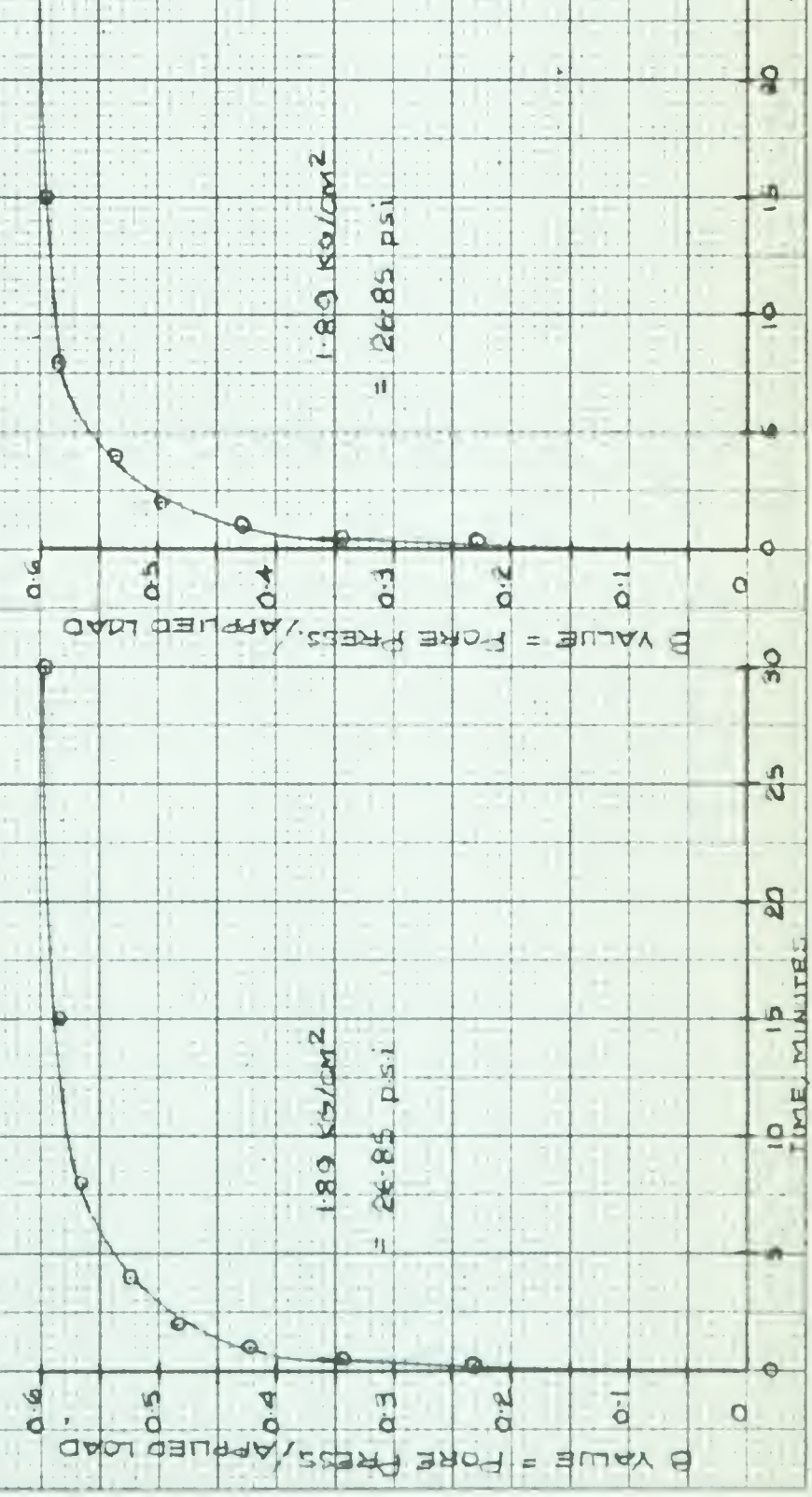
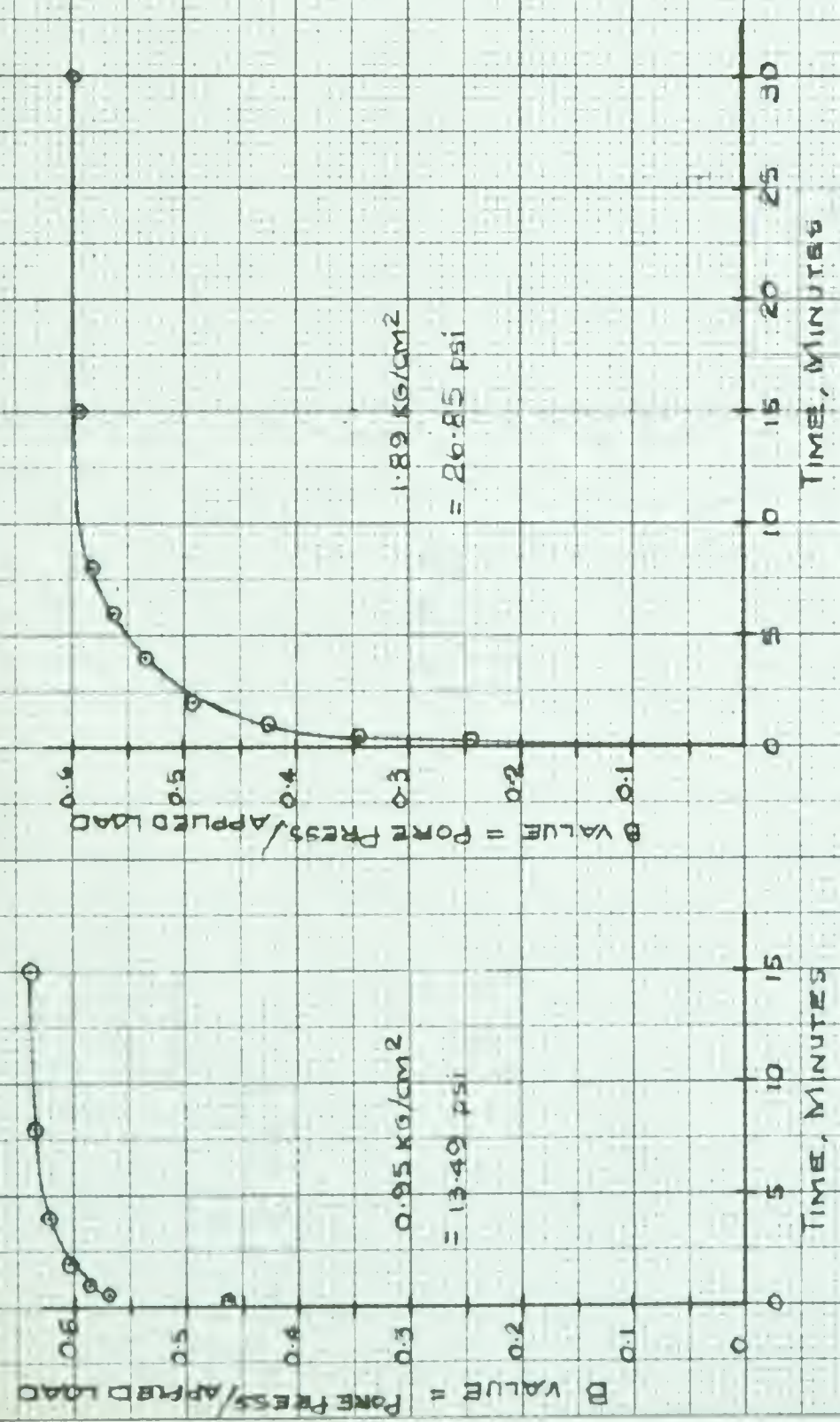
APPENDIX D

D.1 General

The pertinent results of the pore pressure reaction test performed on the sample in Special Consolidation Test number 2 were presented in TABLE VIII. The complete results of this reaction test are presented graphically in this Appendix.

FIGURE D1 presents the graphical results. The B parameter is plotted against time for the reaction to take place.

FIGURE D1
PORE PRESSURE REACTION TEST
B-PARAMETER VS. TIME
SPECIAL CONSOLIDATION TEST NUMBER 2
BEARPAW
SHALE



B29828